THE

ARCHITECT'S AND BUILDER'S POCKET-BOOK.

A HANDBOOK FOR ARCHITECTS, STRUCTURAL ENGINEERS, BUILDERS, AND DRAUGHTSMEN.

BY

FRANK E. KIDDER, C.E., PH.D.,

Author of "Building Construction and Superintendence."

Illustrated with 1000 Engravings, mostly from Original Designs.

FIFTEENTH EDITION, REVISED,

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This Book

IS RESPECTFULLY DEDICATED TO THOSE WHOSE KINDNESS HAS ENABLED ME TO PRODUCE IT.

TO MY PARENTS,

WHO GAVE ME THE EDUCATION UPON WHICH IT IS BASED;

TO MY WIFE,

FOR HER LOVING SYMPATHY, ENCOURAGEMENT, AND ASSISTANCE;

TO ORLANDO W. NORCROSS

OF WORCESTER, MASS.,

WHOSE SUPERIOR PRACTICAL KNOWLEDGE OF ALL THAT PERTAINS TO BUILDING HAS GIVEN ME A MORE INTELLIGENT AND PRACTICAL VIEW OF THE SCIENCE OF CONSTRUCTION THAN I SHOULD OTHERWISE HAVE OBTAINED.*

* Dedication to First Edition.

The publishers and the author will be grateful to any of the readers of this volume who will kindly call their attention to errors, typographical or otherwise, discovered therein, in order that these may be corrected before the next edition goes to press, also for suggestions towards making the index more complete.

JOHN WILEY & SONS.

43 & 45 EAST NINETEENTH STREET,
NEW YORK.

PREFACE TO FIFTEENTH EDITION.

THE changes in this edition consist of the correction of all typographical errors reported to the publishers, and the re-writing of Chapters XXIII and XXIV. This work has been done by Rudolph P. Miller, who was for ten years connected with the Department of Buildings, New York City, and for the last five years as its Chief Engineer. During his connection with the Department of Buildings he had large opportunities for studying fire-proof construction particularly, and gave the subject of Reinforced Concrete much study, drafting the first regulations ever promulgated in this country regarding its use. These regulations have formed the basis of the regulations since adopted by the cities of this country, in many instances the major part of them being copied verbatim.

Chapter XXIII has been revised, one half of the matter in the old edition having been used again. The new matter has been substituted for such parts as have been found unnecessary or out of date.

Chapter XXIV, on Reinforced Concrete, is entirely new, the whole manuscript being original and Mr. Miller's own work.

At the time of Mr. Kidder's revision for the fourteenth edition, he was not altogether satisfied with the chapters on fire-proofing and reinforced concrete, and had he lived, would have revised them personally; and we believe that the chapters as they now stand would meet with his approval were he living.

Professor Alvah H. Sabin has also kindly brought the section on Paints and Varnishes up to date.

PREFACE TO FOURTEENTH EDITION.

It is now nearly twenty years since the author, then quite a young man, completed the first edition of this work, which, although containing but 586 pages, had required about three years for its preparation. At that time the author thought he had covered all of those practical details relating to the planning and construction of buildings, with which the architect was concerned, tolerably well, and it would appear as though the purchasers of the book thought so too, but as the years have come and gone, so many and such great improvements have taken place in the building world, so many articles invented, new methods of construction developed, higher standards established, that the present edition, although containing nearly three times as many pages, is perhaps not more complete, for the times, than was the first edition.

When preparing the first edition, it was the aim of the author to give to architects and builders a handbook which should be, in its field, as useful and reliable as Trautwine's had been to civil engineers; and with that object constantly in view, the book has been revised from time to time to meet the changed conditions in building construction and equipment.

About three years ago it was thought, by the publishers and the author, that a thorough and complete revision of the book should be undertaken, and although the re-writing of a work of this character, even with the thirteenth edition to work from, involved many months of close and constant application, the utilization of those hours which one ordinarily takes for recreation, and at the best more or less interruption to his regular business, and consequent reduction in income, the writer under-

took to prepare a work of a still wider scope, and which should be thoroughly up-to-date in every particular, or at least as far as is practicable, in a work requiring a period of three years in its preparation, and from that time to this he has spared no labor or expense to make the book as useful and complete as he possibly could, without making it too bulky.

In this revision the author has had in view:

1st. A reference-book which should contain some information on every subject (except design) likely to come before an architect, structural engineer, draughtsman, or master-builder, including data for estimating the approximate cost.

2d. To as thoroughly cover the subject of architectural engineering as is practicable in a handbook.

3d. To present all information in as simple and convenient a form for immediate application as is consistent with accuracy. To this end a great many new tables, arranged and computed by the author, have been inserted.

At the time the first edition was written, the term "Architectural Engineering" had not been used in its present application, and the term "Structural Engineering," when used, referred almost exclusively to bridge work.

To-day, structural and architectural engineers are concerned almost exclusively with building construction, and their work is more closely allied to that of the architect than to that of the civil engineer; hence the author has had in mind the needs of the structural engineer and draughtsman as well as those of the architect and builder, and the book should be of nearly equal value to both.

Where it was impossible, for lack of space, to go extensively into any subject, references to other books or sources of information have been given, so that in this way the book may serve as a general index to the many lines of work, materials, and manufactured products entering into the planning, construction, and equipment of buildings.

To attain the objects in view, it has been necessary to add considerably to the number of pages, but as experience has shown that the book is used principally at the desk or draughting-table, and is seldom carried in the pocket, it is believed that the convenience of having everything in one book will more than offset any disadvantage resulting from increase in bulk.

Nearly the entire book has been re-written, and great pains

have been taken to furnish reliable data. A large number of experts in various lines have assisted the author, as is manifest by the foot-notes and references. To all of such, and to the many authors of technical works, and to the publishers of technical journals, who have kindly consented to the use of cuts and data, the author takes pleasure in acknowledging his indebtedness. Also to Mr. E. S. Hand, of New York, who, for many years, has rendered material assistance in collecting data along the line of manufactured products.

The names and addresses of manufacturers have been given solely for the convenience of the users of the book, and not for any pecuniary considerations; in fact, if money considerations had solely appealed to the writer, this book would never have been re-written, because a technical work of this character can never adequately compensate, in money, for the time, labor, and thought required in its preparation. The many words of appreciation which have come to the author from hundreds of those who have found the book useful have been a great stimulus to further increase its usefulness.

As in the former prefaces, the author requests that any one discovering errors in the work or who may have any suggestions looking to the further improvement of the book, will communicate the same to him, that the book may be made as complete and reliable as possible.

Finally, the author desires to acknowledge his indebtedness to the publishers, who have heartily seconded his efforts in every particular, and who have spared no pains or expense to make a perfect handbook.

F. E. Kidder.

Denver, Colo., July 18th, 1904.

PREFACE TO FIRST EDITION, 1884.

In preparing the following pages, it has ever been the aim of the author to give to the architects and builders of this country a *reference book* which should be for them what Trautwine's "Pocket-Book" is to engineers,—a compendium of practical facts, rules, and tables, presented in a form as convenient for application as possible, and as reliable as our present knowledge will permit. Only so much *theory* has been given as will render the application of the formulas more apparent, and aid the student in understanding, in some measure, the principles upon which the formulas are based. It is believed that nothing has been given in this book but what has been borne out in practice.

As this book was not written for *engineers*, the more intricate problems of building construction, which may fairly be said to come within the province of the civil engineer, have been omitted.

Desiring to give as much information as possible likely to be of service to architects and builders, the author has borrowed and quoted from many sources, in most cases with the permission of the authors. Much practical information has been derived from the various handbooks published by the large manufacturers of rolled-iron beams, bars, etc.; and the author has always found the publishers willing to aid him whenever requested.

Although but very little has been taken from Trautwine's "Pocket-Book for Engineers," yet this valuable book has served the author as a model, which he has tried to imitate as well as the difference in the subjects would permit; and if his work shall prove of as much value to architects and builders as Mr. Trautwine's has to engineers, he will feel amply rewarded for his labor.

As it is impossible for the author to verify all of the dimensions

and miscellaneous information contained in Part III., he cannot speak for their accuracy, except that they were in all cases taken from what were considered reliable sources of information. The tables in Part II. have been carefully computed, and it is believed are free from any large errors. There are so many points of information often required by architects and builders, that it is difficult for one person to compile them all; and although the present volume is by no means a small one, yet the author desires to make his work as useful as possible to those for whom it has been prepared, and he will therefore be pleased to receive any information of a serviceable nature pertaining to architecture or building, that it may be inserted in future editions should such become necessary, and for the correction of any errors that may be found.

The author, while compiling this volume, has consulted a great number of works relating to architecture and building; and as he has frequently been asked by students and draughtsmen to refer them to books from which they might acquire a better knowledge of construction and building, the following list of books is given as valuable works on the various subjects indicated by the titles:—

"Notes on Building Construction," compiled for the use of the students in the science and art schools, South Kensington, England. 3 vols. Rivingtons, publishers, London.

"Building Superintendence," by T. M. Clark, architect and professor of architecture, Massachusetts Institute of Technology. J. R. Osgood & Co., publishers, Boston.

"The American House Carpenter" and "The Theory of Transverse Strains," both by Mr. R. G. Hatfield, architect, formerly of New York.

"Graphical Analysis of Roof-Trusses," by Professor Charles E. Green of the University of Michigan.

"The Fire Protection of Mills," by C. J. H. Woodbury, inspector for the Factory Mutual Fire Insurance Companies. John Wiley & Sons, publishers, New York.

"House Drainage and Water Service," by James C. Bayles, editor of "The Iron Age" and "The Metal Worker." David Williams, publisher, New York.

"The Builders' Guide and Estimators' Price-Book," and "Plaster and Plastering, Mortars, and Cements," by Fred. T. Hodgson, editor of "The Builder and Wood Worker." Industrial Publication Company, New York.

"Foundations and Concrete Works," and "Art of Building," by E. Dobson. Weale's Series, London.

It would be well if all of the above books might be found in every architect's office; but if the expense prevents that, the ambitious student and draughtsman should at least make himself acquainted with their contents. These works will also be found of great value to the enterprising builder.

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PART I.

PRACTICAL

ARITHMETIC, GEOMETRY, AND TRIGONOMETRY.

RULES, TABLES, AND PROBLEMS.

PRACTICAL ARITHMETIC AND GEOMETRY.

SIGNS AND CHARACTERS.

THE following signs and characters are generally used to denote and abbreviate the several mathematical operations:—

The sign $=$ means equal to, or equality.

$-$ means minus or less, or subtraction.

$+$ means plus, or addition.

$\times$ means multiplied by, or multiplication.

$\div$ means divided by, or division.

^{2} ^{3} Index or power, meaning that the number to which they are added is to be squared (^{2}) or cubed (^{3}).

: is to, :: so is, : to — Signs of proportion.

$\sqrt{}$ means that the square root of the number before which it is placed is required.

$\sqrt[3]{}$ means that the cube root of the number before which it is placed is required.

—— the *bar* indicates that all the numbers under it are to be taken together.

( ) the *parenthesis* means that all the numbers between are to be taken as one quantity.

. means decimal parts; thus, 2.5 means $2\frac{5}{10}$, 0.46 means $\frac{46}{100}$.

° means degrees, ′ minutes, ″ seconds.

$\therefore$ means hence.

INVOLUTION.

To square a number, multiply the number by itself, and the product will be the square; thus, the square of $18=18\times18=324$.

The cube of a number is the product obtained by multiplying the number by itself, and that product by the number again; thus, the cube of $14=14\times14\times14=2744$.

The fourth power of a number is the product obtained by multiplying the number by itself four times; thus, the fourth power of $10=10\times10\times10\times10=10000$.

EVOLUTION.

Square Root.—Rule for determining the square root of a number.

1st, Divide the given number into periods of two figures each, commencing at the right if it is a whole number, and at the decimal-point if there are decimals; thus, $\dot{1}0\dot{2}3\dot{6}.8\dot{1}2\dot{6}$.

2d, Find the largest square in the left-hand period, and place its root in the quotient; subtract the said square from the left-hand period, and to the remainder bring down the next period for a new dividend.

3d, Double the root already found, and annex one cipher for a trial divisor, see how many times it will go in the dividend, and put the number in the quotient, also, in place of the cipher in the divisor;—multiply this final divisor by the number in the quotient just found, and subtract the product from the dividend, and to the remainder bring down the next period for a new dividend, and proceed as before. If it should be found that the trial divisor cannot be contained in the dividend, bring down the next period for a new dividend, and annex another cipher to the trial divisor, and put a cipher in the quotient, and proceed as before.

EXAMPLE $\dot{1}0\dot{2}3\dot{6}.8\dot{1}2\dot{6}$ (101.17 square root.

1

201) 0236

201

2021) 3581

2021

20227) 156026

141589

14437

Cube Root.—To extract the cube root of a number, point off the number from right to left into periods of three figures each, and, if there is a decimal, commence at the decimal-point, and point off into periods, going both ways.

Ascertain the highest root of the first period, and place to right of number, as in long division; cube the root thus found, and subtract from the first period; to the remainder annex the next period; square the root already found, and multiply by three, and annex two ciphers for the trial divisor. Find how often this trial divisor is contained in the dividend, and write the result in the root.

Add together the trial divisor, three times the product of the first figure of the root by the second with one cipher annexed, and the square of the second figure in the root; multiply the sum by the last figure in the root, and subtract from the dividend; to the remainder annex the next period, and proceed as before.

When the trial divisor is greater than the dividend, write a cipher in the root, annex the next period to the dividend, and proceed as before.

Desired the $\sqrt[3]{493039}$.

	$49\dot{3}03\dot{9}$ (79 cube root.
$7\times7\times7=$	343
$7\times7\times3=14700$	150039
$7\times9\times3=\ \ 1890$	
$9\times9=\ \ \ \ 81$	
16671	150039

Desired the $\sqrt[3]{403583.419}$.

	$40\dot{3}58\dot{3}.41\dot{9}$ (73.9 cube root.
$7\times7\times7=$	343
$7\times7\times3=14700$	60583
$7\times3\times3=\ \ \ 630$	
$3\times3=\ \ \ \ \ 9$	
15339	46017
$73\times73\times3=1598700$	14566419
$73\times\ 9\times3=\ \ \ 19710$	
$9\times9=\ \ \ \ \ 81$	
1618491	14566419

Desired the $\sqrt[3]{158252.632929}$.

	$15\dot{8}25\dot{2}.63\dot{2}92\dot{9}$ (54.09 cube root.
$5\times5\times5=$	125
$5\times5\times3=7500$	33252
$5\times4\times3=\ \ 600$	
$4\times4=\ \ \ 16$	
8116	32464
$540\times540\times3=87480000$	788632929
$540\times\ \ 9\times3=\ \ \ 145800$	
$9\times9=\ \ \ \ \ 81$	
87625881	788632929

TABLE

OF

SQUARES, CUBES, SQUARE ROOTS, CUBE ROOTS, AND RECIPROCALS,

From 1 to 1054.

The following table, taken from Searle's "Field Engineering,' will be found of great convenience in finding the square, cube, square root, cube root, and reciprocal of any number from 1 to 1054. The reciprocal of a number is the quotient obtained by dividing 1 by the number. Thus the reciprocal of 8 is $1 \div 8 = 0.125$.

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
1	1	1	1.0000000	1.0000000	1.000000000
2	4	8	1.4142136	1.2599210	.500000000
3	9	27	1.7320508	1.4422496	.333333333
4	16	64	2.0000000	1.5874011	.250000000
5	25	125	2.2360680	1.7099759	.200000000
6	36	216	2.4494897	1.8171206	.166666667
7	49	343	2.6457513	1.9129312	.142857143
8	64	512	2.8284271	2.0000000	.125000000
9	81	729	3.0000000	2.0800837	.111111111
10	100	1000	3.1622777	2.1544347	.100000000
11	121	1331	3.3166248	2.2239801	.090909091
12	144	1728	3.4641016	2.2894286	.083333333
13	169	2197	3.6055513	2.3513347	.076923077
14	196	2744	3.7416574	2.4101422	.071428571
15	225	3375	3.8729833	2.4662121	.066666667
16	256	4096	4.0000000	2.5198421	.062500000
17	289	4913	4.1231056	2.5712816	.058823529
18	324	5832	4.2426407	2.6207414	.055555556
19	361	6859	4.3588989	2.6684016	.052631579
20	400	8000	4.4721360	2.7144177	.050000000
21	441	9261	4.5825757	2.7589243	.047619048
22	484	10648	4.6904158	2.8020393	.045454545
23	529	12167	4.7958315	2.8438670	.043478261
24	576	13824	4.8989795	2.8844991	.041666667
25	625	15625	5.0000000	2.9240177	.040000000
26	676	17576	5.0990195	2.9624960	.038461538
27	729	19683	5.1961524	3.0000000	.037037037
28	784	21952	5.2915026	3.0365889	.035714286
29	841	24389	5.3851648	3.0723168	.034482759
30	900	27000	5.4772256	3.1072325	.033333333
31	961	29791	5.5677644	3.1413806	.032258065
32	1024	32768	5.6568542	3.1748021	.031250000
33	1089	35937	5.7445626	3.2075343	.030303030
34	1156	39304	5.8309519	3.2396118	.029411765
35	1225	42875	5.9160798	3.2710663	.028571429
36	1296	46656	6.0000000	3.3019272	.027777778
37	1369	50653	6.0827625	3.3322218	.027027027
38	1444	54872	6.1644140	3.3619754	.026315789
39	1521	59319	6.2449980	3.3912114	.025641026
40	1600	64000	6.3245553	3.4199519	.025000000
41	1681	68921	6.4031242	3.4482172	.024390244
42	1764	74088	6.4807407	3.4760266	.023809524
43	1849	79507	6.5574385	3.5033981	.023255814
44	1936	85184	6.6332496	3.5303483	.022727273
45	2025	91125	6.7082039	3.5568933	.022222222
46	2116	97336	6.7823300	3.5830479	.021739130
47	2209	103823	6.8556546	3.6088261	.021276600
48	2304	110592	6.9282032	3.6342411	.020833333
49	2401	117649	7.0000000	3.6593057	.020408163
50	2500	125000	7.0710678	3.6840314	.020000000
51	2601	132651	7.1414284	3.7084298	.019607843
52	2704	140608	7.2111026	3.7325111	.019230769
53	2809	148877	7.2801099	3.7562858	.018867925
54	2916	157464	7.3484692	3.7797631	.018518519
55	3025	166375	7.4161985	3.8029525	.018181818
56	3136	175616	7.4833148	3.8258624	.017857143
57	3249	185193	7.5498344	3.8485011	.017543860
58	3364	195112	7.6157731	3.8708766	.017241379
59	3481	205379	7.6811457	3.8929965	.016949153
60	3600	216000	7.7459667	3.9148676	.016666667
61	3721	226981	7.8102497	3.9364972	.016393443
62	3844	238328	7.8740079	3.9578915	.016129032

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
63	3969	250047	7.9372539	3.9790571	.015873016
64	4096	262144	8.0000000	4.0000000	.015625000
65	4225	274625	8.0622577	4.0207256	.015384615
66	4356	287496	8.1240384	4.0412401	.015151515
67	4489	300763	8.1853528	4.0615480	.014925373
68	4624	314432	8.2462113	4.0816551	.014705882
69	4761	328509	8.3066239	4.1015661	.014492754
70	4900	343000	8.3666003	4.1212853	.014285714
71	5041	357911	8.4261498	4.1408178	.014084507
72	5184	373248	8.4852814	4.1601676	.013888889
73	5329	389017	8.5440037	4.1793390	.013698630
74	5476	405224	8.6023253	4.1983364	.013513514
75	5625	421875	8.6602540	4.2171633	.013333333
76	5776	438976	8.7177979	4.2358236	.013157895
77	5929	456533	8.7749644	4.2543210	.012987013
78	6084	474552	8.8317609	4.2726586	.012820513
79	6241	493039	8.8881944	4.2908404	.012658228
80	6400	512000	8.9442719	4.3088695	.012500000
81	6561	531441	9.0000000	4.3267487	.012345679
82	6724	551368	9.0553851	4.3444815	.012195122
83	6889	571787	9.1104336	4.3620707	.012048193
84	7056	592704	9.1651514	4.3795191	.011904762
85	7225	614125	9.2195445	4.3968296	.011764706
86	7396	636056	9.2736185	4.4140049	.011627907
87	7569	658503	9.3273791	4.4310476	.011494253
88	7744	681472	9.3808315	4.4479602	.011363636
89	7921	704969	9.4339811	4.4647451	.011235955
90	8100	729000	9.4868330	4.4814047	.011111111
91	8281	753571	9.5393920	4.4979414	.010989011
92	8464	778688	9.5916630	4.5143574	.010869565
93	8649	804357	9.6436508	4.5306549	.010752688
94	8836	830584	9.6953597	4.5468359	.010638298
95	9025	857375	9.7467943	4.5629026	.010526316
96	9216	884736	9.7979590	4.5788570	.010416667
97	9409	912673	9.8488578	4.5947009	.010309278
98	9604	941192	9.8994949	4.6104363	.010204082
99	9801	970299	9.9498744	4.6260650	.010101010
100	10000	1000000	10.0000000	4.6415888	.010000000
101	10201	1030301	10.0498756	4.6570095	.009900990
102	10404	1061208	10.0995049	4.6723287	.009803922
103	10609	1092727	10.1488916	4.6875482	.009708738
104	10816	1124864	10.1980390	4.7026694	.009615385
105	11025	1157625	10.2469508	4.7176940	.009523810
106	11236	1191016	10.2956301	4.7326235	.009433962
107	11449	1225043	10.3440804	4.7474594	.009345794
108	11664	1259712	10.3923048	4.7622032	.009259259
109	11881	1295029	10.4403065	4.7768562	.009174312
110	12100	1331000	10.4880885	4.7914199	.009090909
111	12321	1367631	10.5356538	4.8058955	.009009009
112	12544	1404928	10.5830052	4.8202845	.008928571
113	12769	1442897	10.6301458	4.8345881	.008849558
114	12996	1481544	10.6770783	4.8488076	.008771930
115	13225	1520875	10.7238053	4.8629442	.008695652
116	13456	1560896	10.7703296	4.8769990	.008620690
117	13689	1601613	10.8166538	4.8909732	.008547009
118	13924	1643032	10.8627805	4.9048681	.008474576
119	14161	1685159	10.9087121	4.9186847	.008403361
120	14400	1728000	10.9544512	4.9324242	.008333333
121	14641	1771561	11.0000000	4.9460874	.008264463
122	14884	1815848	11.0453610	4.9596757	.008196721
123	15129	1860867	11.0905365	4.9731898	.008130081
124	15376	1906624	11.1355287	4.9866310	.008064516

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
125	15625	1953125	11.1803399	5.0000000	.008000000
126	15876	2000376	11.2249722	5.0132979	.007936508
127	16129	2048383	11.2694277	5.0265257	.007874016
128	16384	2097152	11.3137085	5.0396842	.007812500
129	16641	2146689	11.3578167	5.0527743	.007751938
130	16900	2197000	11.4017543	5.0657970	.007692308
131	17161	2248091	11.4455231	5.0787531	.007633588
132	17424	2299968	11.4891253	5.0916434	.007575758
133	17689	2352637	11.5325626	5.1044687	.007518797
134	17956	2406104	11.5758369	5.1172299	.007462687
135	18225	2460375	11.6189500	5.1299278	.007407407
136	18496	2515456	11.6619038	5.1425632	.007352941
137	18769	2571353	11.7046999	5.1551367	.007299270
138	19044	2628072	11.7473401	5.1676493	.007246377
139	19321	2685619	11.7898261	5.1801015	.007194245
140	19600	2744000	11.8321596	5.1924941	.007142857
141	19881	2803221	11.8743421	5.2048279	.007092199
142	20164	2863288	11.9163753	5.2171034	.007042254
143	20449	2924207	11.9582607	5.2293215	.006993007
144	20736	2985984	12.0000000	5.2414828	.006944444
145	21025	3048625	12.0415946	5.2535879	.006896552
146	21316	3112136	12.0830460	5.2656374	.006849315
147	21609	3176523	12.1243557	5.2776321	.006802721
148	21904	3241792	12.1655251	5.2895725	.006756757
149	22201	3307949	12.2065556	5.3014592	.006711409
150	22500	3375000	12.2474487	5.3132928	.006666667
151	22801	3442951	12.2882057	5.3250740	.006622517
152	23104	3511808	12.3288280	5.3368033	.006578947
153	23409	3581577	12.3693169	5.3484812	.006535948
154	23716	3652264	12.4096736	5.3601084	.006493506
155	24025	3723875	12.4498996	5.3716854	.006451613
156	24336	3796416	12.4899960	5.3832126	.006410256
157	24649	3869893	12.5299641	5.3946907	.006369427
158	24964	3944312	12.5698051	5.4061202	.006329114
159	25281	4019679	12.6095202	5.4175015	.006289308
160	25600	4096000	12.6491106	5.4288352	.006250000
161	25921	4173281	12.6885775	5.4401218	.006211180
162	26244	4251528	12.7279221	5.4513618	.006172840
163	26569	4330747	12.7671453	5.4625556	.006134969
164	26896	4410944	12.8062485	5.4737037	.006097561
165	27225	4492125	12.8452326	5.4848066	.006060606
166	27556	4574296	12.8840987	5.4958647	.006024096
167	27889	4657463	12.9228480	5.5068784	.005988024
168	28224	4741632	12.9614814	5.5178484	.005952381
169	28561	4826809	13.0000000	5.5287748	.005917160
170	28900	4913000	13.0384048	5.5396583	.005882353
171	29241	5000211	13.0766968	5.5504991	.005847953
172	29584	5088448	13.1148770	5.5612978	.005813953
173	29929	5177717	13.1529464	5.5720546	.005780347
174	30276	5268024	13.1909060	5.5827702	.005747126
175	30625	5359375	13.2287566	5.5934447	.005714286
176	30976	5451776	13.2664992	5.6040787	.005681818
177	31329	5545233	13.3041347	5.6146724	.005649718
178	31684	5639752	13.3416641	5.6252263	.005617978
179	32041	5735339	13.3790882	5.6357408	.005586592
180	32400	5832000	13.4164079	5.6462162	.005555556
181	32761	5929741	13.4536240	5.6566528	.005524862
182	33124	6028568	13.4907376	5.6670511	.005494505
183	33489	6128487	13.5277493	5.6774114	.005464481
184	33856	6229504	13.5646600	5.6877340	.005434783
185	34225	6331625	13.6014705	5.6980192	.005405405
186	34596	6434856	13.6381817	5.7082675	.005376344

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
187	34969	6539203	13.6747943	5.7184791	.005347594
188	35344	6644672	13.7113092	5.7286543	.005319149
189	35721	6751269	13.7477271	5.7387936	.005291005
190	36100	6859000	13.7840488	5.7488971	.005263158
191	36481	6967871	13.8202750	5.7589652	.005235602
192	36864	7077888	13.8564065	5.7689982	.005208333
193	37249	7189057	13.8924440	5.7789966	.005181347
194	37636	7301384	13.9283883	5.7889604	.005154639
195	38025	7414875	13.9642400	5.7988900	.005128205
196	38416	7529536	14.0000000	5.8087857	.005102041
197	38809	7645373	14.0356688	5.8186479	.005076142
198	39204	7762392	14.0712473	5.8284767	.005050505
199	39601	7880599	14.1067360	5.8382725	.005025126
200	40000	8000000	14.1421356	5.8480355	.005000000
201	40401	8120601	14.1774469	5.8577660	.004975124
202	40804	8242408	14.2126704	5.8674643	.004950495
203	41209	8365427	14.2478068	5.8771307	.004926108
204	41616	8489664	14.2828569	5.8867653	.004901961
205	42025	8615125	14.3178211	5.8963685	.004878049
206	42436	8741816	14.3527001	5.9059406	.004854369
207	42849	8869743	14.3874946	5.9154817	.004830918
208	43264	8998912	14.4222051	5.9249921	.004807692
209	43681	9129329	14.4568323	5.9344721	.004784689
210	44100	9261000	14.4913767	5.9439220	.004761905
211	44521	9393931	14.5258390	5.9533418	.004739336
212	44944	9528128	14.5602198	5.9627320	.004716981
213	45369	9663597	14.5945195	5.9720926	.004694836
214	45796	9800344	14.6287388	5.9814240	.004672897
215	46225	9938375	14.6628783	5.9907264	.004651163
216	46656	10077696	14.6969385	6.0000000	.004629630
217	47089	10218313	14.7309199	6.0092450	.004608295
218	47524	10360232	14.7648231	6.0184617	.004587156
219	47961	10503459	14.7986486	6.0276502	.004566210
220	48400	10648000	14.8323970	6.0368107	.004545455
221	48841	10793861	14.8660687	6.0459435	.004524887
222	49284	10941048	14.8996644	6.0550489	.004504505
223	49729	11089567	14.9331845	6.0641270	.004484305
224	50176	11239424	14.9666295	6.0731779	.004464286
225	50625	11390625	15.0000000	6.0822020	.004444444
226	51076	11543176	15.0332964	6.0911994	.004424779
227	51529	11697083	15.0665192	6.1001702	.004405286
228	51984	11852352	15.0996689	6.1091147	.004385965
229	52441	12008989	15.1327460	6.1180332	.004366812
230	52900	12167000	15.1657509	6.1269257	.004347826
231	53361	12326391	15.1986842	6.1357924	.004329004
232	53824	12487168	15.2315462	6.1446337	.004310345
233	54289	12649337	15.2643375	6.1534495	.004291845
234	54756	12812904	15.2970585	6.1622401	.004273504
235	55225	12977875	15.3297097	6.1710058	.004255319
236	55696	13144256	15.3622915	6.1797466	.004237288
237	56169	13312053	15.3948043	6.1884628	.004219409
238	56644	13481272	15.4272486	6.1971544	.004201681
239	57121	13651919	15.4596248	6.2058218	.004184100
240	57600	13824000	15.4919334	6.2144650	.004166667
241	58081	13997521	15.5241747	6.2230843	.004149378
242	58564	14172488	15.5563492	6.2316797	.004132231
243	59049	14348907	15.5884573	6.2402515	.004115226
244	59536	14526784	15.6204994	6.2487998	.004098361
245	60025	14706125	15.6524758	6.2573248	.004081633
246	60516	14886936	15.6843871	6.2658266	.004065041
247	61009	15069223	15.7162336	6.2743054	.004048583
248	61504	15252992	15.7480157	6.2827613	.004032258

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
249	62001	15438249	15.7797338	6.2911946	.004016064
250	62500	15625000	15.8113883	6.2996053	.004000000
251	63001	15813251	15.8429795	6.3079935	.003984064
252	63504	16003008	15.8745079	6.3163596	.003968254
253	64009	16194277	15.9059737	6.3247035	.003952569
254	64516	16387064	15.9373775	6.3330256	.003937008
255	65025	16581375	15.9687194	6.3413257	.003921569
256	65536	16777216	16.0000000	6.3496042	.003906250
257	66049	16974593	16.0312195	6.3578611	.003891051
258	66564	17173512	16.0623784	6.3660968	.003875969
259	67081	17373979	16.0934769	6.3743111	.003861004
260	67600	17576000	16.1245155	6.3825043	.003846154
261	68121	17779581	16.1554944	6.3906765	.003831418
262	68644	17984728	16.1864141	6.3988279	.003816794
263	69169	18191447	16.2172747	6.4069585	.003802281
264	69696	18399744	16.2480768	6.4150687	.003787879
265	70225	18609625	16.2788206	6.4231583	.003773585
266	70756	18821096	16.3095064	6.4312276	.003759398
267	71289	19034163	16.3401346	6.4392767	.003745318
268	71824	19248832	16.3707055	6.4473057	.003731343
269	72361	19465109	16.4012195	6.4553148	.003717472
270	72900	19683000	16.4316767	6.4633041	.003703704
271	73441	19902511	16.4620776	6.4712736	.003690037
272	73984	20123648	16.4924225	6.4792236	.003676471
273	74529	20346417	16.5227116	6.4871541	.003663004
274	75076	20570824	16.5529454	6.4950653	.003649635
275	75625	20796875	16.5831240	6.5029572	.003636364
276	76176	21024576	16.6132477	6.5108300	.003623188
277	76729	21253933	16.6433170	6.5186839	.003610108
278	77284	21484952	16.6733320	6.5265189	.003597122
279	77841	21717639	16.7032931	6.5343351	.003584229
280	78400	21952000	16.7332005	6.5421326	.003571429
281	78961	22188041	16.7630546	6.5499116	.003558719
282	79524	22425768	16.7928556	6.5576722	.003546099
283	80089	22665187	16.8226038	6.5654144	.003533569
284	80656	22906304	16.8522995	6.5731385	.003521127
285	81225	23149125	16.8819430	6.5808443	.003508772
286	81796	23393656	16.9115345	6.5885323	.003496503
287	82369	23639903	16.9410743	6.5962023	.003484321
288	82944	23887872	16.9705627	6.6038545	.003472222
289	83521	24137569	17.0000000	6.6114890	.003460208
290	84100	24389000	17.0293864	6.6191060	.003448276
291	84681	24642171	17.0587221	6.6267054	.003436426
292	85264	24897088	17.0880075	6.6342874	.003424658
293	85849	25153757	17.1172428	6.6418522	.003412969
294	86436	25412184	17.1464282	6.6493998	.003401361
295	87025	25672375	17.1755640	6.6569302	.003389831
296	87616	25934336	17.2046505	6.6644437	.003378378
297	88209	26198073	17.2336879	6.6719403	.003367003
298	88804	26463592	17.2626765	6.6794200	.003355705
299	89401	26730899	17.2916165	6.6868831	.003344482
300	90000	27000000	17.3205081	6.6943295	.003333333
301	90601	27270901	17.3493516	6.7017593	.003322259
302	91204	27543608	17.3781472	6.7091729	.003311258
303	91809	27818127	17.4068952	6.7165700	.003300330
304	92416	28094464	17.4355958	6.7239508	.003289474
305	93025	28372625	17.4642492	6.7313155	.003278689
306	93636	28652616	17.4928557	6.7386641	.003267974
307	94249	28934443	17.5214155	6.7459967	.003257329
308	94864	29218112	17.5499288	6.7533134	.003246753
309	95481	29503629	17.5783958	6.7606143	.003236246
310	96100	29791000	17.6068169	6.7678995	.003225806

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
311	96721	30080231	17.6351921	6.7751690	.003215434
312	97344	30371328	17.6635217	6.7824229	.003205128
313	97969	30664297	17.6918060	6.7896613	.003194888
314	98596	30959144	17.7200451	6.7968844	.003184713
315	99225	31255875	17.7482393	6.8040921	.003174603
316	99856	31554496	17.7763888	6.8112847	.003164557
317	100489	31855013	17.8044938	6.8184620	.003154574
318	101124	32157432	17.8325545	6.8256242	.003144654
319	101761	32461759	17.8605711	6.8327714	.003134796
320	102400	32768000	17.8885438	6.8399037	.003125000
321	103041	33076161	17.9164729	6.8470213	.003115265
322	103684	33386248	17.9443584	6.8541240	.003105590
323	104329	33698267	17.9722008	6.8612120	.003095975
324	104976	34012224	18.0000000	6.8682855	.003086420
325	105625	34328125	18.0277564	6.8753443	.003076923
326	106276	34645976	18.0554701	6.8823888	.003067485
327	106929	34965783	18.0831413	6.8894188	.003058104
328	107584	35287552	18.1107703	6.8964345	.003048780
329	108241	35611289	18.1383571	6.9034359	.003039514
330	108900	35937000	18.1659021	6.9104232	.003030303
331	109561	36264691	18.1934054	6.9173964	.003021148
332	110224	36594368	18.2208672	6.9243556	.003012048
333	110889	36926037	18.2482876	6.9313008	.003003003
334	111556	37259704	18.2756669	6.9382321	.002994012
335	112225	37595375	18.3030052	6.9451496	.002985075
336	112896	37933056	18.3303028	6.9520533	.002976190
337	113569	38272753	18.3575598	6.9589434	.002967359
338	114244	38614472	18.3847763	6.9658198	.002958580
339	114921	38958219	18.4119526	6.9726826	.002949853
340	115600	39304000	18.4390889	6.9795321	.002941176
341	116281	39651821	18.4661853	6.9863681	.002932551
342	116964	40001688	18.4932420	6.9931906	.002923977
343	117649	40353607	18.5202592	7.0000000	.002915452
344	118336	40707584	18.5472370	7.0067962	.002906977
345	119025	41063625	18.5741756	7.0135791	.002898551
346	119716	41421736	18.6010752	7.0203490	.002890173
347	120409	41781923	18.6279360	7.0271058	.002881844
348	121104	42144192	18.6547581	7.0338497	.002873563
349	121801	42508549	18.6815417	7.0405806	.002865330
350	122500	42875000	18.7082869	7.0472987	.002857143
351	123201	43243551	18.7349940	7.0540041	.002849003
352	123904	43614208	18.7616630	7.0606967	.002840909
353	124609	43986977	18.7882942	7.0673767	.002832861
354	125316	44361864	18.8148877	7.0740440	.002824859
355	126025	44738875	18.8414437	7.0806988	.002816901
356	126736	45118016	18.8679623	7.0873411	.002808989
357	127449	45499293	18.8944436	7.0939709	.002801120
358	128164	45882712	18.9208879	7.1005885	.002793216
359	128881	46268279	18.9472953	7.1071937	.002785595
360	129600	46656000	18.9736660	7.1137866	.002777778
361	130321	47045881	19.0000000	7.1203674	.002770083
632	131044	47437928	19.0262976	7.1269360	.002762431
363	131769	47832147	19.0525589	7.1334925	.002754821
364	132496	48228544	19.0787840	7.1400370	.002747253
365	133225	48627125	19.1049732	7.1465695	.002739726
366	133956	49027896	19.1311265	7.1530901	.002732240
367	134689	49430863	19.1572441	7.1595988	.002724796
368	135424	49836032	19.1833261	7.1660957	.002717391
369	136161	50243409	19.2093727	7.1725809	.002710027
370	136900	50653000	19.2353841	7.1790544	.002702703
371	137641	51064811	19.2613603	7.1855162	.002695418
372	138384	51478848	19.2873015	7.1919663	.002688172

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
373	139129	51895117	19.3132079	7.1984050	.002680965
374	139876	52313624	19.3390796	7.2048322	.002673797
375	140625	52734375	19.3649167	7.2112479	.002666667
376	141376	53157376	19.3907194	7.2176522	.002659574
377	142129	53582633	19.4164878	7.2240450	002652520
378	142884	54010152	19.4422221	7.2304268	.002645503
379	143641	54439939	19.4679223	7.2367972	.002638522
380	144400	54872000	19.4935887	7.2431565	.002631579
381	145161	55306341	19.5192213	7.2495045	.002624672
382	145924	55742968	19.5448203	7.2558415	.002617801
383	146689	56181887	19.5703858	7.2621675	.002610966
384	147456	56623104	19.5959179	7.2684824	.002604167
385	148225	57066625	19.6214169	7.2747864	.002597403
386	148996	57512456	19.6468827	7.2810794	.002590674
387	149769	57960603	19.6723156	7.2873617	.002583979
388	150544	58411072	19.6977156	7.2936330	.002577320
389	151321	58863869	19.7230829	7.2998936	.002570694
390	152100	59319000	19.7484177	7.3061436	.002564103
391	152881	59776471	19.7737199	7.3123828	.002557545
392	153664	60236288	19.7989899	7.3186114	.002551020
393	154449	60698457	19.8242276	7.3248295	.002544529
394	155236	61162984	19.8494332	7.3310369	.002538071
395	156025	61629875	19.8746069	7.3372339	.002531646
396	156816	62099136	19.8997487	7.3434205	.002525253
397	157609	62570773	19.9248588	7.3495966	.002518892
398	158404	63044792	19.9499373	7.3557624	.002512563
399	159201	63521199	19.9749844	7.3619178	.002506266
400	160000	64000000	20.0000000	7.3680630	.002500000
401	160801	64481201	20.0249844	7.3741979	.002493766
402	161604	64964808	20.0499377	7.3803227	.002487562
403	162409	65450827	20.0748599	7.3864373	.002481390
404	163216	65939264	20.0997512	7.3925418	.002475248
405	164025	66430125	20.1246118	7.3986363	.002469136
406	164836	66923416	20.1494417	7.4047206	.002463054
407	165649	67419143	20.1742410	7.4107950	.002457002
408	166464	67917312	20.1990099	7.4168595	.002450980
409	167281	68417929	20.2237484	7.4229142	.002444988
410	168100	68921000	20.2484567	7.4289589	.002439024
411	168921	69426531	20.2731349	7.4349938	.002433090
412	169744	69934528	20.2977831	7.4410189	.002427184
413	170569	70444997	20.3224014	7.4470342	.002421308
414	171396	70957944	20.3469899	7.4530399	.002415459
415	172225	71473375	20.3715488	7.4590359	.002409639
416	173056	71991296	20.3960781	7.4650223	.002403846
417	173889	72511713	20.4205779	7.4709991	.002398082
418	174724	73034632	20.4450483	7.4769664	.002392344
419	175561	73560059	20.4694895	7.4829242	.002386635
420	176400	74088000	20.4939015	7.4888724	.002380952
421	177241	74618461	20.5182845	7.4948113	.002375297
422	178084	75151448	20.5426386	7.5007406	.002369668
423	178929	75686967	20.5669638	7.5066607	.002364066
424	179776	76225024	20.5912603	7.5125715	.002358491
425	180625	76765625	20.6155281	7.5184730	.002352941
426	181476	77308776	20.6397674	7.5243652	.002347418
427	182329	77854483	20.6639783	7.5302482	.002341920
428	183184	78402752	20.6881609	7.5361221	.002336449
429	184041	78953589	20.7123152	7.5419867	.002331002
430	184900	79507000	20.7364414	7.5478423	.002325581
431	185761	80062991	20.7605395	7.5536888	.002320186
432	186624	80621568	20.7846097	7.5595263	.002314815
433	187489	81182737	20.8086520	7.5653548	.002309469
434	188356	81746504	20.8326667	7.5711743	.002304147

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
435	189225	82312875	20.8566536	7.5769849	.002298851
436	190096	82881856	20.8806130	7.5827865	.002293578
437	190969	83453453	20.9045450	7.5885793	.002288330
438	191844	84027672	20.9284495	7.5943633	.002283105
439	192721	84604519	20.9523268	7.6001385	.002277904
440	193600	85184000	20.9761770	7.6059049	.002272727
441	194481	85766121	21.0000000	7.6116626	.002267574
442	195364	86350888	21.0237960	7.6174116	.002262443
443	196249	86938307	21.0475652	7.6231519	.002257336
444	197136	87528384	21.0713075	7.6288837	.002252252
445	198025	88121125	21.0950231	7.6346067	.002247191
446	198916	88716536	21.1187121	7.6403213	.002242152
447	199809	89314623	21.1423745	7.6460272	.002237136
448	200704	89915392	21.1660105	7.6517247	.002232143
449	201601	90518849	21.1896201	7.6574138	.002227171
450	202500	91125000	21.2132034	7.6630943	.002222222
451	203401	91733851	21.2367606	7.6687665	.002217295
452	204304	92345408	21.2602916	7.6744303	.002212389
453	205209	92959677	21.2837967	7.6800857	.002207506
454	206116	93576664	21.3072758	7.6857328	.002202643
455	207025	94196375	21.3307290	7.6913717	.002197802
456	207936	94818816	21.3541565	7.6970023	.002192982
457	208849	95443993	21.3775583	7.7026246	.002188184
458	209764	96071912	21.4009346	7.7082388	.002183406
459	210681	96702579	21.4242853	7.7138448	.002178649
460	211600	97336000	21.4476106	7.7194426	.002173913
461	212521	97972181	21.4709106	7.7250325	.002169197
462	213444	98611128	21.4941853	7.7306141	.002164502
463	214369	99252847	21.5174348	7.7361877	.002159827
464	215296	99897344	21.5406592	7.7417532	.002155172
465	216225	100544625	21.5638587	7.7473109	.002150538
466	217156	101194696	21.5870331	7.7528606	.002145923
467	218089	101847563	21.6101828	7.7584023	.002141328
468	219024	102503232	21.6333077	7.7639361	.002136752
469	219961	103161709	21.6564078	7.7694620	.002132196
470	220900	103823000	21.6794834	7.7749801	.002127660
471	221841	104487111	21.7025344	7.7804904	.002123142
472	222784	105154048	21.7255610	7.7859928	.002118644
473	223729	105823817	21.7485632	7.7914875	.002114165
474	224676	106496424	21.7715411	7.7969745	.002109705
475	225625	107171875	21.7944947	7.8024538	.002105263
476	226576	107850176	21.8174242	7.8079254	.002100840
477	227529	108531333	21.8403297	7.8133892	.002096436
478	228484	109215352	21.8632111	7.8188456	.002092050
479	229441	109902239	21.8860686	7.8242942	.002087683
480	230400	110592000	21.9089023	7.8297353	.002083333
481	231361	111284641	21.9317122	7.8351688	.002079002
482	232324	111980168	21.9544984	7.8405949	.002074689
483	233289	112678587	21.9772610	7.8460134	.002070393
484	234256	113379904	22.0000000	5.8514244	.002066116
485	235225	114084125	22.0227155	7.8568281	.002061856
486	236196	114791256	22.0454077	7.8622242	.002057613
487	237169	115501303	22.0680765	7.8676130	.002053388
488	238144	116214272	22.0907220	7.8729944	.002049180
489	239121	116930169	22.1133444	7.8783684	.002044990
490	240100	117649000	22.1359436	7.8837352	.002040816
491	241081	118370771	22.1585198	7.8890946	.002036660
492	242064	119095488	22.1810730	7.8944468	.002032520
493	243049	119823157	22.2036033	7.8997917	.002028398
494	244036	120553784	22.2261108	7.9051294	.002024291
495	245025	121287375	22.2485955	7.9104599	.002020202
496	246016	122023936	22.2710575	7.9157832	.002016129

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
497	247009	122763473	22.2934968	7.9210994	.002012072
498	248004	123505992	22.3159136	7.9264085	.002008032
499	249001	124251499	22.3383079	7.9317104	.002004008
500	250000	125000000	22.3606798	7.9370053	.002000000
501	251001	125751501	22.3830293	7.9422931	.001996008
502	252004	126506008	22.4053565	7.9475739	.001992032
503	253009	127263527	22.4276615	7.9528477	.001988072
504	254016	128024064	22.4499443	7.9581144	.001984127
505	255025	128787625	22.4722051	7.9633743	.001980198
506	256036	129554216	22.4944438	7.9686271	.001976285
507	257049	130323843	22.5166605	7.9738731	.001972387
508	258064	131096512	22.5388553	7.9791122	.001968504
509	259081	131872229	22.5610283	7.9843444	.001964637
510	260100	132651000	22.5831796	7.9895697	.001960784
511	261121	133432831	22.6053091	7.9947883	.001956947
512	262144	134217728	22.6274170	8.0000000	.001953125
513	263169	135005697	22.6495033	8.0052049	.001949318
514	264196	135796744	22.6715681	8.0104032	.001945525
515	265225	136590875	22.6936114	8.0155946	.001941748
516	266256	137388096	22.7156334	8.0207794	.001937984
517	267289	138188413	22.7376340	8.0259574	.001934236
518	268324	138991832	22.7596134	8.0311287	.001930502
519	269361	139798359	22.7815715	8.0362935	.001926782
520	270400	140608000	22.8035085	8.0414515	.001923077
521	271441	141420761	22.8254244	8.0466030	.001919386
522	272484	142236648	22.8473193	8.0517479	.001915709
523	273529	143055667	22.8691933	8.0568862	.001912046
524	274576	143877824	22.8910463	8.0620180	.001908397
525	275625	144703125	22.9128785	8.0671432	.001904762
526	276676	145531576	22.9346899	8.0722620	.001901141
527	277729	146363183	22.9564806	8.0773743	.001897533
528	278784	147197952	22.9782506	8.0824800	.001893939
529	279841	148035889	23.0000000	8.0875794	.001890359
530	280900	148877000	23.0217289	8.0926723	.001886792
531	281961	149721291	23.0434372	8.0977589	.001883239
532	283024	150568768	23.0651252	8.1028390	.001879699
533	284089	151419437	23.0867928	8.1079128	.001876173
534	285156	152273304	23.1084400	8.1129803	.001872659
535	286225	153130375	23.1300670	8.1180414	.001869159
536	287296	153990656	23.1516738	8.1230962	.001865672
537	288369	154854153	23.1732605	8.1281447	.001862197
538	289444	155720872	23.1948270	8.1331870	.001858736
539	290521	156590819	23.2163735	8.1382230	.001855288
540	291600	157464000	23.2379001	8.1432529	.001851852
541	292681	158340421	23.2594067	8.1482765	.001848429
542	293764	159220088	23.2808935	8.1532939	.001845018
543	294849	160103007	23.3023604	8.1583051	.001841621
544	295936	160989184	23.3238076	8.1633102	.001838235
545	297025	161878625	23.3452351	8.1683092	.001834862
546	298116	162771336	23.3666429	8.1733020	.001831502
547	299209	163667323	23.3880311	8.1782888	.001828154
548	300304	164566592	23.4093998	8.1832695	.001824818
549	301401	165469149	23.4307490	8.1882441	.001821494
550	302500	166375000	23.4520788	8.1932127	.001818182
551	303601	167284151	23.4733892	8.1981753	.001814882
552	304704	168196608	23.4946802	8.2031319	.001811594
553	305809	169112377	23.5159520	8.2080825	.001808318
554	306916	170031464	23.5372046	8.2130271	.001805054
555	308025	170953875	23.5584380	8.2179657	.001801802
556	309136	171879616	23.5796522	8.2228985	.001798561
557	310249	172808693	23.6008474	8.2278254	.001795332
558	311364	173741112	23.6220236	8.2327463	.001792115

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
559	312481	174676879	23.6431808	8.2376614	.001788909
560	313600	175616000	23.6643191	8.2425706	.001785714
561	314721	176558481	23.6854386	8.2474740	.001782531
562	315844	177504328	23.7065392	8.2523715	.001779359
563	316969	178453547	23.7276210	8.2572633	.001776199
564	318096	179406144	23.7486842	8.2621492	.001773050
565	319225	180362125	23.7697286	8.2670294	.001769912
566	320356	181321496	23.7907545	8.2719039	.001766784
567	321489	182284263	23.8117618	8.2767726	.001763668
568	322624	183250432	23.8327506	8.2816355	.001760563
569	323761	184220000	23.8537209	8.2864928	.001757469
570	324900	185193000	23.8746728	8.2913444	.001754386
571	326041	186169411	23.8956063	8.2961903	.001751313
572	327184	187149248	23.9165215	8.3010304	.001748252
573	328329	188132517	23.9374184	8.3058651	.001745201
574	329476	189119224	23.9582971	8.3106941	.001742160
575	330625	190109375	23.9791576	8.3155175	.001739130
576	331776	191102976	24.0000000	8.3203353	.001736111
577	332929	192100033	24.0208243	8.3251475	.001733102
578	334084	193100552	24.0416306	8.3299542	.001730104
579	335241	194104539	24.0624188	8.3347553	.001727116
580	336400	195112000	24.0831891	8.3395509	.001724138
581	337561	196122941	24.1039416	8.3443410	.001721170
582	338724	197137368	24.1246762	8.3491256	.001718213
583	339889	198155287	24.1453929	8.3539047	.001715266
584	341056	199176704	24.1660919	8.3586784	.001712329
585	342225	200201625	24.1867732	8.3634466	.001709402
586	343396	201230056	24.2074369	8.3682095	.001706485
587	344569	202262003	24.2280829	8.3729668	.001703578
588	345744	203297472	24.2487113	8.3777188	.001700680
589	346921	204336469	24.2693222	8.3824653	.001697793
590	348100	205379000	24.2899156	8.3872065	.001694915
591	349281	206425071	24.3104916	8.3919423	.001692047
592	350464	207474688	24.3310501	8.3966729	.001689189
593	351649	208527857	24.3515913	8.4013981	.001686341
594	352836	209584584	24.3721152	8.4061180	.001683502
595	354025	210644875	24.3926218	8.4108326	.001680672
596	355216	211708736	24.4131112	8.4155419	.001677852
597	356409	212776173	24.4335834	8.4202460	.001675042
598	357604	213847192	24.4540385	8.4249448	.001672241
599	358801	214921799	24.4744765	8.4296383	.001669449
600	360000	216000000	24.4948974	8.4343267	.001666667
601	361201	217081801	24.5153013	8.4390098	.001663894
602	362404	218167208	24.5356883	8.4436877	.001661130
603	363609	219256227	24.5560583	8.4483605	.001658375
604	364816	220348864	24.5764115	8.4530281	.001655629
605	366025	221445125	24.5967478	8.4576906	.001652893
606	367236	222545016	24.6170673	8.4623479	.001650165
607	368449	223648543	24.6373700	8.4670001	.001647446
608	369664	224755712	24.6576560	8.4716471	.001644737
609	370881	225866529	24.6779254	8.4762892	.001642036
610	372100	226981000	24.6981781	8.4809261	.001639344
611	373321	228099131	24.7184142	8.4855579	.001636661
612	374544	229220928	24.7386338	8.4901848	.001633987
613	375769	230346397	24.7588368	8.4948065	.001631321
614	376996	231475544	24.7790234	8.4994233	.001628664
615	378225	232608375	24.7991935	8.5040350	.001626016
616	379456	233744896	24.8193473	8.5086417	.001623377
617	380689	234885113	24.8394847	8.5132435	.001620746
618	381924	236029032	24.8596058	8.5178403	.001618123
619	383161	237176659	24.8797106	8.5224321	.001615509
620	384400	238328000	24.8997992	8.5270189	.001612903

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
621	385641	239483061	24.9198716	8.5316009	.001610306
622	386884	240641848	24.9399278	8.5361780	.001607717
623	388129	241804367	24.9599679	8.5407501	.001605136
624	389376	242970624	24.9799920	8.5453173	.001602564
625	390625	244140625	25.0000000	8.5498797	.001600000
626	391876	245314376	25.0199920	8.5544372	.001597444
627	393129	246491883	25.0399681	8.5589899	.001594896
628	394384	247673152	25.0599282	8.5635377	.001592357
629	395641	248858189	25.0798724	8.5680807	.001589825
630	396900	250047000	25.0998008	8.5726189	.001587302
631	398161	251239591	25.1197134	8.5771523	.001584786
632	399424	252435968	25.1396102	8.5816809	.001582278
633	400689	253636137	25.1594913	8.5862047	.001579779
634	401956	254840104	25.1793566	8.5907238	.001577287
635	403225	256047875	25.1992063	8.5952380	.001574803
636	404496	257259456	25.2190404	8.5997476	.001572327
637	405769	258474853	25.2388589	8.6042525	.001569859
638	407044	259694072	25.2586619	8.6087526	.001567398
639	408321	260917119	25.2784493	8.6132480	.001564945
640	409600	262144000	25.2982213	8.6177388	.001562500
641	410881	263374721	25.3179778	8.6222248	.001560062
642	412164	264609288	25.3377189	8.6267063	.001557632
643	413449	265847707	25.3574447	8.6311830	.001555210
644	414736	267089984	25.3771551	8.6356551	.001552795
645	416025	268336125	25.3968502	8.6401226	.001550388
646	417316	269586136	25.4165301	8.6445855	.001547988
647	418609	270840023	25.4361947	8.6490437	.001545595
648	419904	272097792	25.4558441	8.6534974	.001543210
649	421201	273359449	25.4754784	8.6579465	.001540832
650	422500	274625000	25.4950976	8.6623911	.001538462
651	423801	275894451	25.5147016	8.6668310	.001536098
652	425104	277167808	25.5342907	8.6712665	.001533742
653	426409	278445077	25.5538647	8.6756974	.001531394
654	427716	279726264	25.5734237	8.6801237	.001529052
655	429025	281011375	25.5929678	8.6845456	.001526718
656	430336	282300416	25.6124969	8.6889630	.001524390
657	431649	283593393	25.6320112	8.6933759	.001522070
658	432964	284890312	25.6515107	8.6977843	.001519757
659	434281	286191179	25.6709953	8.7021882	.001517451
660	435600	287496000	25.6904652	8.7065877	.001515152
661	436921	288804781	25.7099203	8.7109827	.001512859
662	438244	290117528	25.7293607	8.7153734	.001510574
663	439569	291434247	25.7487864	8.7197596	.001508296
664	440896	292754944	25.7681975	8.7241414	.001506024
665	442225	294079625	25.7875939	8.7285187	.001503759
666	443556	295408296	25.8069758	8.7328918	.001501502
667	444889	296740963	25.8263431	8.7372604	.001499250
668	446224	298077632	25.8456960	8.7416246	.001497006
669	447561	299418309	25.8650343	8.7459846	.001494768
670	448900	300763000	25.8843582	8.7503401	.001492537
671	450241	302111711	25.9036677	8.7546913	.001490313
672	451584	303464448	25.9229628	8.7590383	.001488095
673	452929	304821217	25.9422435	8.7633809	.001485884
674	454276	306182024	25.9615100	8.7677192	.001483680
675	455625	307546875	25.9807621	8.7720532	.001481481
676	456976	308915776	26.0000000	8.7763830	.001479290
677	458329	310288733	26.0192237	8.7807084	.001477105
678	459684	311665752	26.0384331	8.7850296	.001474926
679	461041	313046839	26.0576284	8.7893466	.004472754
680	462400	314432000	26.0768096	8.7936593	.001470588
681	463761	315821241	26.0959767	8.7979679	.001468429
682	465124	317214568	26.1151297	8.8022721	.001466276

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
683	466489	318611987	26.1342687	8.8065722	.001464129
684	467856	320013504	26.1533937	8.8108681	.001461988
685	469225	321419125	26.1725047	8.8151598	.001459854
686	470596	322828856	26.1916017	8.8194474	.001457726
687	471969	324242703	26.2106848	8.8237307	.001455604
688	473344	325660672	26.2297541	8.8280099	.001453488
689	474721	327082769	26.2488095	8.8322850	.001451379
690	476100	328509000	26.2678511	8.8365559	.001449275
691	477481	329939371	26.2868789	8.8408227	.001447178
692	478864	331373888	26.3058929	8.8450854	.001445087
693	480249	332812557	26.3248932	8.8493440	.001443001
694	481636	334255384	26.3438797	8.8535985	.001440922
695	483025	335702375	26.3628527	8.8578489	.001438849
696	484416	337153536	26.3818119	8.8620952	.001436782
697	485809	338608873	26.4007576	8.8663375	.001434720
698	487204	340068392	26.4196896	8.8705757	.001432665
699	488601	341532099	26.4386081	8.8748099	.001430615
700	490000	343000000	26.4575131	8.8790400	.001428571
701	491401	344472101	26.4764046	8.8832661	.001426534
702	492804	345948408	26.4952826	8.8874882	.001424501
703	494209	347428927	26.5141472	8.8917063	.001422475
704	495616	348913664	26.5329983	8.8959204	.001420455
705	497025	350402625	26.5518361	8.9001304	.001418440
706	498436	351895816	26.5706605	8.9043366	.001416431
707	499849	353393243	26.5894716	8.9085387	.001414427
708	501264	354894912	26.6082694	8.9127369	.001412429
709	502681	356400829	26.6270539	8.9169311	.001410437
710	504100	357911000	26.6458252	8.9211214	.001408451
711	505521	359425431	26.6645833	8.9253078	.001406470
712	506944	360944128	26.6833281	8.9294902	.001404494
713	508369	362467097	26.7020598	8.9336687	.001402525
714	509796	363994344	26.7207784	8.9378433	.001400560
715	511225	365525875	26.7394839	8.9420140	.001398601
716	512656	367061696	26.7581763	8.9461809	.001396648
717	514089	368601813	26.7768557	8.9503438	.001394700
718	515524	370146232	26.7955220	8.9545029	.001392758
719	516961	371694959	26.8141754	8.9586581	.001390821
720	518400	373248000	26.8328157	8.9628095	.001388889
721	519841	374805361	26.8514432	8.9669570	.001386963
722	521284	376367048	26.8700577	8.9711007	.001385042
723	522729	377933067	26.8886593	8.9752406	.001383126
724	524176	379503424	26.9072481	8.9793766	.001381215
725	525625	381078125	26.9258240	8.9835089	.001379310
726	527076	382657176	26.9443872	8.9876373	.001377410
727	528529	384240583	26.9629375	8.9917620	.001375516
728	529984	385828352	26.9814751	8.9958829	.001373626
729	531441	387420489	27.0000000	9.0000000	.001371742
730	532900	389017000	27.0185122	9.0041134	.001369863
731	534361	390617891	27.0370117	9.0082229	.001367989
732	535824	392223168	27.0554985	9.0123288	.001366120
733	537289	393832837	27.0739727	9.0164309	.001364256
734	538756	395446904	27.0924344	9.0205293	.001362398
735	540225	397065375	27.1108834	9.0246239	.001360544
736	541696	398688256	27.1293199	9.0287149	.001358696
737	543169	400315553	27.1477439	9.0328021	.001356852
738	544644	401947272	27.1661554	9.0368857	.001355014
739	546121	403583419	27.1845544	9.0409655	.001353180
740	547600	405224000	27.2029410	9.0450419	.001351351
741	549081	406869021	27.2213152	9.0491142	001349528
742	550564	408518488	27.2396769	9.0531831	001347709
743	552049	410172407	27.2580263	9.0572482	001345895
744	553536	411830784	27.2763634	9.0613098	001344086

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
745	555025	413493625	27.2946881	9.0653677	.001342282
746	556516	415160936	27.3130006	9.0694220	.001340483
747	558009	416832723	27.3313007	9.0734726	.001338688
748	559504	418508992	27.3495887	9.0775197	.001336898
749	561001	420189749	27.3678644	9.0815631	.001335113
750	562500	421875000	27.3861279	9.0856030	.001333333
751	564001	423564751	27.4043792	9.0896392	.001331558
752	565504	425259008	27.4226184	9.0936719	.001329787
753	567009	426957777	27.4408455	9.0977010	.001328021
754	568516	428661064	27.4590604	9.1017265	.001326260
755	570025	430368875	27.4772633	9.1057485	.001324503
756	571536	432081216	27.4954542	9.1097669	.001322751
757	573049	433798093	27.5136330	9.1137818	.001321004
758	574564	435519512	27.5317998	9.1177931	.001319261
759	576081	437245479	27.5499546	9.1218010	.001317523
760	577600	438976000	27.5680975	9.1258053	.001315789
761	579121	440711081	27.5862284	9.1298061	.001314060
762	580644	442450728	27.6043475	9.1338034	.001312336
763	582169	444194947	27.6224546	9.1377971	.001310616
764	583696	445943744	27.6405499	9.1417874	.001308901
765	585225	447697125	27.6586334	9.1457742	.001307190
766	586756	449455096	27.6767050	9.1497576	.001305483
767	588289	451217663	27.6947648	9.1537375	.001303781
768	589824	452984832	27.7128129	9.1577139	.001302083
769	591361	454756609	27.7308492	9.1616869	.001300390
770	592900	456533000	27.7488739	9.1656565	.001298701
771	594441	458314011	27.7668808	9.1696225	.001297017
772	595984	460099648	27.7848880	9.1735852	.001295337
773	597529	461889917	27.8028775	9.1775445	.001293661
774	599076	463684824	27.8208555	9.1815003	.001291990
775	600625	465484375	27.8388218	9.1854527	.001290323
776	602176	467288576	27.8567766	9.1894018	.001288660
777	603729	469097433	27.8747197	9.1933474	.001287001
778	605284	470910952	27.8926514	9.1972897	.001285347
779	606841	472729139	27.9105715	9.2012286	.001283697
780	608400	474552000	27.9284801	9.2051641	.001282051
781	609961	476379541	27.9463772	9.2090962	.001280410
782	611524	478211768	27.9642629	9.2130250	.001278772
783	613089	480048687	27.9821372	9.2169505	.001277139
784	614656	481890304	28.0000000	9.2208726	.001275510
785	616225	483736625	28.0178515	9.2247914	.001273885
786	617796	485587656	28.0356915	9.2287068	.001272265
787	619369	487443403	28.0535203	9.2326189	.001270648
788	620944	489303872	28.0713377	9.2365277	.001269036
789	622521	491169069	28.0891438	9.2404333	.001267427
790	624100	493039000	28.1069386	9.2443355	.001265823
791	625681	494913671	28.1247222	9.2482344	.001264223
792	627264	496793088	28.1424946	9.2521300	.001262626
793	628849	498677257	28.1602557	9.2560224	.001261034
794	630436	500566184	28.1780056	9.2599114	.001259446
795	632025	502459875	28.1957444	9.2637973	.001257862
796	633616	504358336	28.2134720	9.2676798	.001256281
797	635209	506261573	28.2311884	9.2715592	.001254705
798	636804	508169592	28.2488938	9.2754352	.001253133
799	638401	510082399	28.2665881	9.2793081	.001251564
800	640000	512000000	28.2842712	9.2831777	.001250000
801	641601	513922401	28.3019434	9.2870440	.001248439
802	643204	515849608	28.3196045	9.2909072	.001246883
803	644809	517781627	28.3372546	9.2947671	.001245330
804	646416	519718464	28.3548938	9.2986239	.001243781
805	648025	521660125	28.3725219	9.3024775	.001242236
806	649636	523606616	28.3901391	9.3063278	.001240695

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
807	651249	525557943	28.4077454	9.3101750	.001239157
808	652864	527514112	28.4253408	9.3140190	.001237624
809	654481	529475129	28.4429253	9.3178599	.001236094
810	656100	531441000	28.4604989	9.3216975	.001234568
811	657721	533411731	28.4780617	9.3255320	.001233046
812	659344	535387328	28.4956137	9.3293634	.001231527
813	660969	537367797	28.5131549	9.3331916	.001230012
814	662596	539353144	28.5306852	9.3370167	.001228501
815	664225	541343375	28.5482048	9.3408386	.001226994
816	665856	543338496	28.5657137	9.3446575	.001225490
817	667489	545338513	28.5832119	9.3484731	.001223990
818	669124	547343432	28.6006993	9.3522857	.001222494
819	670761	549353259	28.6181760	9.3560952	.001221001
820	672400	551368000	28.6356421	9.3599016	.001219512
821	674041	553387661	28.6530976	9.3637049	.001218027
822	675684	555412248	28.6705424	9.3675051	.001216545
823	677329	557441767	28.6879766	9.3713022	.001215067
824	678976	559476224	28.7054002	9.3750963	.001213592
825	680625	561515625	28.7228132	9.3788873	.001212121
826	682276	563559976	28.7402157	9.3826752	.001210654
827	683929	565609283	28.7576077	9.3864600	.001209190
828	685584	567663552	28.7749891	9.3902419	.001207729
829	687241	569722789	28.7923601	9.3940206	.001206273
830	688900	571787000	28.8097206	9.3977964	.001204819
831	690561	573856191	28.8270706	9.4015691	.001203369
832	692224	575930368	28.8444102	9.4053387	.001201923
833	693889	578009537	28.8617394	9.4091054	.001200480
834	695556	580093704	28.8790582	9.4128690	.001199041
835	697225	582182875	28.8963666	9.4166297	.001197605
836	698896	584277056	28.9136646	9.4203873	.001196172
837	700569	586376263	28.9309523	9.4241420	.001194743
838	702244	588480472	28.9482297	9.4278936	.001193317
839	703921	590589719	28.9654967	9.4316423	.001191895
840	705600	592704000	28.9827535	9.4353880	.001190476
841	707281	594823321	29.0000000	9.4391307	.001189061
842	708964	596947688	29.0172363	9.4428704	.001187648
843	710649	599077107	29.0344623	9.4466072	.001186240
844	712336	601211584	29.0516781	9.4503410	.001184834
845	714025	603351125	29.0688837	9.4540719	.001183432
846	715716	605495736	29.0860791	9.4577999	.001182033
847	717409	607645423	29.1032644	9.4615249	.001180638
848	719104	609800192	29.1204396	9.4652470	.001179245
849	720801	611960049	29.1376046	9.4689661	.001177856
850	722500	614125000	29.1547595	9.4726824	.001176471
851	724201	616295051	29.1719043	9.4763957	.001175088
852	725904	618470208	29.1890390	9.4801061	.001173709
853	727609	620650477	29.2061637	9.4838136	.001172333
854	729316	622835864	29.2232784	9.4875182	.001170960
855	731025	625026375	29.2403830	9.4912200	.001169591
856	732736	627222016	29.2574777	9.4949188	.001168224
857	734449	629422793	29.2745623	9.4986147	.001166861
858	736164	631628712	29.2916370	9.5023078	.001165501
859	737881	633839779	29.3087018	9.5059980	.001164144
860	739600	636056000	29.3257566	9.5096854	.001162791
861	741321	638277381	29.3428015	9.5133699	.001161440
862	743044	640503928	29.3598365	9.5170515	.001160093
863	744769	642735647	29.3768616	9.5207303	.001158749
864	746496	644972544	29.3938769	9.5244063	.001157407
865	748225	647214625	29.4108823	9.5280794	.001156069
866	749956	649461896	29.4278779	9.5317497	.001154734
867	751689	651714363	29.4448637	9.5354172	.001153403
868	753424	653972032	29.4618397	9.5390818	.001152074

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
869	755161	656234909	29.4788059	9.5427437	.001150748
870	756900	658503000	29.4957624	9.5464027	.001149425
871	758641	660776311	29.5127091	9.5500589	.001148106
872	760384	663054848	29.5296461	9.5537123	.001146789
873	762129	665338617	29.5465734	9.5573630	.001145475
874	763876	667627624	29.5634910	9.5610108	.001144165
875	765625	669921875	29.5803989	9.5646559	.001142857
876	767376	672221376	29.5972972	9.5682982	.001141553
877	769129	674526133	29.6141858	9.5719377	.001140251
878	770884	676836152	29.6310648	9.5755745	.001138952
879	772641	679151439	29.6479342	9.5792085	.001137656
880	774400	681472000	29.6647939	9.5828397	.001136364
881	776161	683797841	29.6816442	9.5864682	.001135074
882	777924	686128968	29.6984848	9.5900939	.001133787
883	779689	688465387	29.7153159	9.5937169	.001132503
884	781456	690807104	29.7321375	9.5973373	.001131222
885	783225	693154125	29.7489496	9.6009548	.001129944
886	784996	695506456	29.7657521	9.6045696	.001128668
887	786769	697864103	29.7825452	9.6081817	.001127396
888	788544	700227072	29.7993289	9.6117911	.001126126
889	790321	702595369	29.8161030	9.6153977	.001124859
890	792100	704969000	29.8328678	9.6190017	.001123596
891	793881	707347971	29.8496231	9.6226030	.001122334
892	795664	709732288	29.8663690	9.6262016	.001121076
893	797449	712121957	29.8831056	9.6297975	.001119821
894	799236	714516984	29.8998328	9.6333907	.001118568
895	801025	716917375	29.9165506	9.6369812	.001117318
896	802816	719323136	29.9332591	9.6405690	.001116071
897	804609	721734273	29.9499583	9.6441542	.001114827
898	806404	724150792	29.9666481	9.6477367	.001113586
899	808201	726572699	29.9833287	9.6513166	.001112347
900	810000	729000000	30.0000000	9.6548938	.001111111
901	811801	731432701	30.0166620	9.6584684	.001109878
902	813604	733870808	30.0333148	9.6620403	.001108647
903	815409	736314327	30.0499584	9.6656096	.001107420
904	817216	738763264	30.0665928	9.6691762	.001106195
905	819025	741217625	30.0832179	9.6727403	.001104972
906	820836	743677416	30.0998339	9.6763017	.001103753
907	822649	746142643	30.1164407	9.6798604	.001102536
908	824464	748613312	30.1330383	9.6834166	.001101322
909	826281	751089429	30.1496269	9.6869701	.001100110
910	828100	753571000	30.1662063	9.6905211	.001098901
911	829921	756058031	30.1827765	9.6940694	.001097695
912	831744	758550528	30.1993377	9.6976151	.001096491
913	833569	761048497	30.2158899	9.7011583	.001095290
914	835396	763551944	30.2324329	9.7046989	.001094092
915	837225	766060875	30.2489669	9.7082369	.001092896
916	839056	768575296	30.2654919	9.7117723	.001091703
917	840889	771095213	30.2820079	9.7153051	.001090513
918	842724	773620632	30.2985148	9.7188354	.001089325
919	844561	776151559	30.3150128	9.7223631	.001088139
920	846400	778688000	30.3315018	9.7258883	.001086957
921	848241	781229961	30.3479818	9.7294109	.001085776
922	850084	783777448	30.3644529	9.7329309	.001084599
923	851929	786330467	30.3809151	9.7364484	.001083423
924	853776	788889024	30.3973683	9.7399634	.001082251
925	855625	791453125	30.4138127	9.7434758	.001081081
926	857476	794022776	30.4302481	9.7469857	.001079914
927	859329	796597983	30.4466747	9.7504930	.001078749
928	861184	799178752	30.4630924	9.7539979	.001077586
929	863041	801765089	30.4795013	9.7575002	.001076426
930	864900	804357000	30.4959014	9.7610001	.001075269

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
931	866761	806954491	30.5122926	9.7644974	.001074114
932	868624	809557568	30.5286750	9.7679922	.001072961
933	870489	812166237	30.5450487	9.7714845	.001071811
934	872356	814780504	30.5614136	9.7749743	.001070664
935	874225	817400375	30.5777697	9.7784616	.001069519
936	876096	820025856	30.5941171	9.7819466	.001068376
937	877969	822656953	30.6104557	9.7854288	.001067236
938	879844	825293672	30.6267857	9.7889087	.001066098
939	881721	827936019	30.6431069	9.7923861	.001064963
940	883600	830584000	30.6594194	9.7958611	.001063830
941	885481	833237621	30.6757233	9.7993336	.001062699
942	887364	835896888	30.6920185	9.8028036	.001061571
943	889249	838561807	30.7083051	9.8062711	.001060445
944	891136	841232384	30.7245830	9.8097362	.001059322
945	893025	843908625	30.7408523	9.8131989	.001058201
946	894916	846590536	30.7571130	9.8166591	.001057082
947	896809	849278123	30.7733651	9.8201169	.001055966
948	898704	851971392	30.7896086	9.8235723	.001054852
949	900601	854670349	30.8058436	9.8270252	.001053741
950	902500	857375000	30.8220700	9.8304757	.001052632
951	904401	860085351	30.8382879	9.8339238	.001051525
952	906304	862801408	30.8544972	9.8373695	.001050420
953	908209	865523177	30.8706981	9.8408127	.001049318
954	910116	868250664	30.8868904	9.8442536	.001048218
955	912025	870983875	30.9030743	9.8476920	.001047120
956	913936	873722816	30.9192497	9.8511280	.001046025
957	915849	876467493	30.9354166	9.8545617	.001044932
958	917764	879217912	30.9515751	9.8579929	.001043841
959	919681	881974079	30.9677251	9.8614218	.001042753
960	921600	884736000	30.9838668	9.8648483	.001041667
961	923521	887503681	31.0000000	9.8682724	.001040583
962	925444	890277128	31.0161248	9.8716941	.001039501
963	927369	893056347	31.0322413	9.8751135	.001038422
964	929296	895841344	31.0483494	9.8785305	.001037344
965	931225	898632125	31.0644491	9.8819451	.001036269
966	933156	901428696	31.0805405	9.8853574	.001035197
967	935089	904231063	31.0966236	9.8887673	.001034126
968	937024	907039232	31.1126984	9.8921749	.001033058
969	938961	909853209	31.1287648	9.8955801	.001031992
970	940900	912673000	31.1448230	9.8989830	.001030928
971	942841	915498611	31.1608729	9.9023835	.001029866
972	944784	918330048	31.1769145	9.9057817	.001028807
973	946729	921167317	31.1929479	9.9091776	.001027749
974	948676	924010424	31.2089731	9.9125712	.001026694
975	950625	926859375	31.2249900	9.9159624	.001025641
976	952576	929714176	31.2409987	9.9193513	.001024590
977	954529	932574833	31.2569992	9.9227379	.001023541
978	956484	935441352	31.2729915	9.9261222	.001022495
979	958441	938313739	31.2889757	9.9295042	.001021450
980	960400	941192000	31.3049517	9.9328839	.001020408
981	962361	944076141	31.3209195	9.9362613	.001019368
982	964324	946966168	31.3368792	9.9396363	.001018330
983	966289	949862087	31.3528303	9.9430092	.001017294
984	968256	952763904	31.3687743	9.9463797	.001016260
985	970225	955671625	31.3847097	9.9497479	.001015228
986	972196	958585256	31.4006369	9.9531138	.001014199
987	974169	961504803	31.4165561	9.9564775	.001013171
988	976144	964430272	31.4324673	9.9598389	.001012146
989	978121	967361669	31.4483704	9.9631981	.001011122
990	980100	970299000	31.4642654	9.9665549	.001010101
991	982081	973242271	31.4801525	9.9699095	.001009082
992	984064	976191488	31.4960315	9.9732619	.001008065

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
993	986049	979146657	31.5119025	9.9766120	.001007049
994	988036	982107784	31.5277655	9.9799599	.001006036
995	990025	985074875	31.5436206	9.9833055	.001005025
996	992016	988047936	31.5594677	9.9866488	.001004016
997	994009	991026973	31.5753068	9.9899900	.001003009
998	996004	994011992	31.5911380	9.9933289	.001002004
999	998001	997002999	31.6069613	9.9966656	.001001001
1000	1000000	1000000000	31.6227766	10.0000000	.001000000
1001	1002001	1003003001	31.6385840	10.0033322	.0009990010
1002	1004004	1006012008	31.6543836	10.0066622	.0009980040
1003	1006009	1009027027	31.6701752	10.0099899	.0009970090
1004	1008016	1012048064	31.6859590	10.0133155	.0009960159
1005	1010025	1015075125	31.7017349	10.0166389	.0009950249
1006	1012036	1018108216	31.7175030	10.0199601	.0009940358
1007	1014049	1021147343	31.7332633	10.0232791	.0009930487
1008	1016064	1024192512	31.7490157	10.0265958	.0009920635
1009	1018081	1027243729	31.7647603	10.0299104	.0009910803
1010	1020100	1030301000	31.7804972	10.0332228	.0009900990
1011	1022121	1033364331	31.7962262	10.0365330	.0009891197
1012	1024144	1036433728	31.8119474	10.0398410	.0009881423
1013	1026169	1039509197	31.8276609	10.0431469	.0009871668
1014	1028196	1042590744	31.8433666	10.0464506	.0009861933
1015	1030225	1045678375	31.8590646	10.0497521	.0009852217
1016	1032256	1048772096	31.8747549	10.0530514	.0009842520
1017	1034289	1051871913	31.8904374	10.0563485	.0009832842
1018	1036324	1054977832	31.9061123	10.0596435	.0009823183
1019	1038361	1058089859	31.9217794	10.0629364	.0009813543
1020	1040400	1061208000	31.9374388	10.0662271	.0009803922
1021	1042441	1064332261	31.9530906	10.0695156	.0009794319
1022	1044484	1067462648	31.9687347	10.0728020	.0009784736
1023	1046529	1070599167	31.9843712	10.0760863	.0009775171
1024	1048576	1073741824	32.0000000	10.0793684	.0009765625
1025	1050625	1076890625	32.0156212	10.0826484	.0009756098
1026	1052676	1080045576	32.0312348	10.0859262	.0009746589
1027	1054729	1083206683	32.0468407	10.0892019	.0009737098
1028	1056784	1086373952	32.0624391	10.0924755	.0009727626
1029	1058841	1089547389	32.0780298	10.0957469	.0009718173
1030	1060900	1092727000	32.0936131	10.0990163	.0009708738
1031	1062961	1095912791	32.1091887	10.1022835	.0009699321
1032	1065024	1099104768	32.1247568	10.1055487	.0009689922
1033	1067089	1102302937	32.1403173	10.1088117	.0009680542
1034	1069156	1105507304	32.1558704	10.1120726	.0009671180
1035	1071225	1108717875	32.1714159	10.1153314	.0009661836
1036	1073296	1111934656	32.1869539	10.1185882	.0009652510
1037	1075369	1115157653	32.2024844	10.1218428	.0009643202
1038	1077444	1118386872	32.2180074	10.1250953	.0009633911
1039	1079521	1121622319	32.2335229	10.1283457	.0009624639
1040	1081600	1124864000	32.2490310	10.1315941	.0009615385
1041	1083681	1128111921	32.2645316	10.1348403	.0009606148
1042	1085764	1131366088	32.2800248	10.1380845	.0009596929
1043	1087849	1134626507	32.2955105	10.1413266	.0009587738
1044	1089936	1137893184	32.3109888	10.1445667	.0009578544
1045	1092025	1141166125	32.3264598	10.1478047	.0009569378
1046	1094116	1144445336	32.3419233	10.1510406	.0009560229
1047	1096209	1147730823	32.3573794	10.1542744	.0009551098
1048	1098304	1151022592	32.3728281	10.1575062	.0009541985
1049	1100401	1154320649	32.3882695	10.1607359	.0009532888
1050	1102500	1157625000	32.4037035	10.1639636	.0009523810
1051	1104601	1160935651	32.4191301	10.1671893	.0009514748
1052	1106704	1164252608	32.4345495	10.1704129	.0009505703
1053	1108809	1167575877	32.4499615	10.1736344	.0009496676
1054	1110916	1170905464	32.4653662	10.1768539	.0009487666

WEIGHTS AND MEASURES.

Measures of Length.

12 inches = 1 foot.
3 feet = 1 yard = 36 inches.
5½ yards = 1 rod = 198 inches = 16½ ft.
40 rods = 1 furlong = 7920 inches = 660 ft. = 220 yds.
8 furlongs = 1 mile = 63360 inches = 5280 ft. = 1760 yds. = [320 rods.
1 yard = 0.0005682 of a mile.

GUNTER'S CHAIN.

7.92 inches = 1 link.
100 links = 1 chain = 4 rods = 66 feet.
80 chains = 1 mile.

ROPES AND CABLES.

6 feet = 1 fathom. 120 fathoms = 1 cable's length.

TABLE SHOWING INCHES EXPRESSED IN DECIMALS OF A FOOT.

In.	0	1	2	3	4	5	6	7	8	9	10	11
0	Foot	.0833	.1667	.2500	.3333	.4167	.5000	.5833	.6667	.7500	.8333	.9167
1-32	.0026	.0859	.1693	.2526	.3359	.4193	.5026	.5859	.6693	.7526	.8359	.9193
1-16	.0052	.0885	.1719	.2552	.3385	.4219	.5052	.5885	.6719	.7552	.8385	.9219
3-32	.0078	.0911	.1745	.2578	.3411	.4245	.5078	.5911	.6745	.7578	.8411	.9245
1-8	.0104	.0938	.1771	.2604	.3438	.4271	.5104	.5938	.6771	.7604	.8438	.9271
5-32	.0130	.0964	.1797	.2630	.3464	.4297	.5130	.5964	.6797	.7630	.8464	.9297
3-16	.0156	.0990	.1823	.2656	.3490	.4323	.5156	.5990	.6823	.7656	.8490	.9323
7-32	.0182	.1016	.1849	.2682	.3516	.4349	.5182	.6016	.6849	.7682	.8516	.9349
1-4	.0208	.1042	.1875	.2708	.3542	.4375	.5208	.6042	.6875	.7708	.8542	.9375
9-32	.0234	.1068	.1901	.2734	.3568	.4401	.5234	.6068	.6901	.7734	.8568	.9401
5-16	.0260	.1094	.1927	.2760	.3594	.4427	.5260	.6094	.6927	.7760	.8594	.9427
11-32	.0286	.1120	.1953	.2786	.3620	.4453	.5286	.6120	.6953	.7786	.8620	.9453
3-8	.0313	.1146	.1979	.2813	.3646	.4479	.5313	.6146	.6979	.7813	.8646	.9479
13-32	.0339	.1172	.2005	.2839	.3672	.4505	.5339	.6172	.7005	.7839	.8672	.9505
7-16	.0365	.1198	.2031	.2865	.3698	.4531	.5365	.6198	.7031	.7865	.8698	.9531
15-32	.0391	.1224	.2057	.2891	.3724	.4557	.5391	.6224	.7057	.7891	.8724	.9557
1-2	.0417	.1250	.2083	.2917	.3750	.4583	.5417	.6250	.7083	.7917	.8750	.9583
17-32	.0443	.1276	.2109	.2943	.3776	.4609	.5443	.6276	.7109	.7943	.8776	.9609
9-16	.0469	.1302	.2135	.2969	.3802	.4635	.5469	.6302	.7135	.7969	.8802	.9635
19-32	.0495	.1328	.2161	.2995	.3828	.4661	.5495	.6328	.7161	.7995	.8828	.9661
5-8	.0521	.1354	.2188	.3021	.3854	.4688	.5521	.6354	.7188	.8021	.8854	.9688
21-32	.0547	.1380	.2214	.3047	.3880	.4714	.5547	.6380	.7214	.8047	.8880	.9714
11-16	.0573	.1406	.2240	.3073	.3906	.4740	.5573	.6406	.7240	.8073	.8906	.9740
23-32	.0599	.1432	.2266	.3099	.3932	.4766	.5599	.6432	.7266	.8099	.8932	.9766
3-4	.0625	.1458	.2292	.3125	.3958	.4792	.5625	.6458	.7292	.8125	.8958	.9792
25-32	.0651	.1484	.2318	.3151	.3984	.4818	.5651	.6484	.7318	.8151	.8984	.9818
13-16	.0677	.1510	.2344	.3177	.4010	.4844	.5677	.6510	.7344	.8177	.9010	.9844
27-32	.0703	.1536	.2370	.3203	.4036	.4870	.5703	.6536	.7370	.8203	.9036	.9870
7-8	.0729	.1563	.2396	.3229	.4063	.4896	.5729	.6563	.7396	.8229	.9063	.9896
29-32	.0755	.1589	.2422	.3255	.4089	.4922	.5755	.6589	.7422	.8255	.9089	.9922
15-16	.0781	.1615	.2448	.3281	.4115	.4948	.5781	.6615	.7448	.8281	.9115	.9948
31-32	.0807	.1641	.2474	.3307	.4141	.4974	.5807	.6641	.7474	.8307	.9141	.9974
	0	1	2	3	4	5	6	7	8	9	10	11

DECIMAL EQUIVALENTS FOR FRACTIONS OF AN INCH.

$\frac{1}{32}$ds.	$\frac{1}{64}$ths.	Decimal.	Fraction.	$\frac{1}{32}$ds.	$\frac{1}{64}$ths.	Decimal.	Fraction.
	1	.015625			33	.515625	
1	2	.03125		17	34	.53125	
	3	.046875			35	.546875	
2	4	.0625	1/16	18	36	.5625	9/16
	5	.078125			37	.578125	
3	6	.09375		19	38	.59375	
	7	.109375			39	.609375	
4	8	.125	1/8	20	40	.625	5/8
	9	.140625			41	.640625	
5	10	.15625		21	42	.65625	
	11	.171875			43	.671875	
6	12	.1875	3/16	22	44	.6875	11/16
	13	.203125			45	.703125	
7	14	.21875		23	46	.71875	
	15	.234375			47	.734375	
8	16	.25	1/4	24	48	.75	3/4
	17	.265625			49	.765625	
9	18	.28125		25	50	.78125	
	19	.296875			51	.796875	
10	20	.3125	5/16	26	52	.8125	13/16
	21	.328125			53	.828125	
11	22	.34375		27	54	.84375	
	23	.359375			55	.859375	
12	24	.375	3/8	28	56	.875	7/8
	25	.390625			57	.890625	
13	26	.40625		29	58	.90625	
	27	.421875			59	.921875	
14	28	.4375	7/16	30	60	.9375	15/16
	29	.453125			61	.953125	
15	30	.46875		31	62	.96875	
	31	.484375			63	.984375	
16	32	.5	1/2	32	64	1.	1

NAUTICAL MEASURE.

A nautical or sea mile is the length of a minute of longitude of the earth at the equator at the level of the sea. It is assumed at 6086.07 feet=1.152664 statute or land miles by the United States Coast Survey.

3 nautical miles=1 league.

MISCELLANEOUS.

1 palm=3 inches.	1 span =9 inches.
1 hand=4 inches.	1 meter=3.2809 feet.

Measures of Surface.

144 square inches=1 square foot.
9 square feet =1 square yard=1296 square inches.
100 square feet =1 square (architects' measure).

LAND.

$30\frac{1}{4}$ square yards	=1 square rod.	
40 square rods	=1 square rood	=1210 square yards.
4 square roods } 10 square chains }	=1 acre =160 square rods.	=4840 square yards.
640 acres	=1 square mile	=3097600 square yards=[102400 sq. rods=2560 sq. roods.
208.71 feet square	=1 acre.	

A *section* of land is a square mile, and a *quarter-section* is 160 acres.

Measures of Volume.

1 gallon liquid measure = 231 cubic inches, and contains 8.339 avoirdupois pounds of distilled water at 39.8° F., or 58333 grains.

1 cubic foot contains 7.48 liquid gallons, or 6.428 dry gallons.

1 gallon dry measure=268.8 cubic inches.

1 bushel (*Winchester*) contains 2150.42 cubic inches, or 77.627 pounds distilled water at 39.8° F.

A heaped bushel contains 2747.715 cubic inches.

DRY.

2 pints =1 quart = 67.2 cubic inches.
4 quarts =1 gallon = 8 pints= 268.8 cubic inches.
2 gallons=1 peck =16 pints= 8 quarts=537.6 cubic inches.
4 pecks =1 bushel=64 pints=32 quarts=8 gals. = 2150.42 [cu. in.
1 cord of wood=128 cubic feet.

LIQUID.

4 gills	= 1 pint	= 16 fluid ounces.	
2 pints	= 1 quart	= 8 gills	= 32 fluid ounces.
4 quarts	= 1 gallon	= 32 gills = 8 pints	= 128 fluid ounces.

In the United States and Great Britain 1 barrel of wine or brandy = 31½ gallons, and contains 4.211 cubic feet.

A hogshead is 63 gallons, but this term is often applied to casks of various capacities.

Cubic Measure.

1728 cubic inches = 1 cubic foot.
27 cubic feet = 1 cubic yard.

In measuring wood, a pile of wood cut 4 feet long, piled 4 feet high, and 8 feet on the ground, making 128 cubic feet, is called a cord.

16 cubic feet make one cord foot.

A *perch of stone* is nominally 16½ feet long, 1 foot high, and 1½ feet thick, and contains 24¾ cubic feet.

A perch of stone is, however, often computed differently in different localities; thus, in most if not all of the States and Territories west of the Mississippi, stone masons figure rubble by the perch of 16½ cu. ft. In Philadelphia, 22 cu. ft. are called a perch. In Chicago, stone is measured by the cord of 100 cu. ft.

A *ton* of shipping is 42 cubic feet in Great Britain and 40 cubic feet in the United States.

Fluid Measure.

60 minims	= 1 fluid drachm.
8 fluid drachms	= 1 ounce.
16 ounces	= 1 pint.
8 pints	= 1 gallon.

Miscellaneous.

Butt of Sherry = 108 gals.	Puncheon of Brandy, 110 to 120 gals.
Pipe of Port = 115 gals.	Puncheon of Rum, 100 to 110 gals.
Butt of Malaga = 105 gals.	Hogshead of Brandy, 55 to 60 gals.
Puncheon of Scotch Whiskey, 110 to 130 gals.	Hogshead of Claret, 46 gals.

Measures of Weight.

The standard avoirdupois pound is the weight of 27.7015 cubic inches of distilled water weighed in air at 39.83°, the barometer at 30 inches; it contains 7000 grains. One pound avoirdupois = 1.2153 pounds troy.

Avoirdupois, or Ordinary Commercial Weight.

1 drachm	= 27.343 grains.	
16 drachms	= 1 ounce	(oz.).
16 ounces	= 1 pound	(lb.).
100 pounds	= 1 hundred weight	(cwt.).
20 hundred weight	= 1 ton.	

In collecting duties upon foreign goods at the United States custom-houses, and also in freighting coal, and selling it by wholesale,—

28 pounds	= 1 quarter.
4 quarters, or 112 lbs.	= 1 hundred weight.
20 hundred weight	= 1 long ton = 2240 pounds.
A stone	= 14 pounds.
A quintal	= 100 pounds.

The following measures are sanctioned by custom or law: 1 bushel = 1.244 cubic feet or $1\frac{1}{4}$ cubic feet nearly.

32 pounds of oats	= 1 bushel.
45 pounds of Timothy-seed	= 1 bushel.
48 pounds of barley	= 1 bushel.
56 pounds of rye	= 1 bushel.
56 pounds of Indian corn	= 1 bushel.
50 pounds of Indian meal	= 1 bushel.
60 pounds of wheat	= 1 bushel.
60 pounds of clover-seed	= 1 bushel.
60 pounds of potatoes	= 1 bushel.
56 pounds of butter	= 1 firkin.
100 pounds of meal or flour	= 1 sack.
100 pounds of grain or flour	= 1 cental.
100 pounds of dry fish	= 1 quintal.
100 pounds of nails	= 1 cask.
196 pounds of flour	= 1 barrel.
200 pounds of beef or pork	= 1 barrel.
80 pounds of lime	= 1 bushel.

Troy Weight.

USED IN WEIGHING GOLD OR SILVER.

24 grains	= 1 pennyweight	(pwt.).
20 pennyweights	= 1 ounce	(oz.).
12 ounces	= 1 pound	(lb.).

A *carat* of the jewellers, for precious stones, is, in the United States, 3.2 grains: in London, 3.17 grains, in Paris, 3.18 grains

are divided into 4 jewellers' grains. In troy, apothecaries', and avoirdupois weights, the grain is the same, one pound troy being equal to .82286 pound avoirdupois.

Apothecaries' Weight.

USED IN COMPOUNDING MEDICINES, AND IN PUTTING UP MEDICAL PRESCRIPTIONS.

20 grains (gr.) = 1 scruple (℈).	8 drachms = 1 ounce (oz.).
3 scruples = 1 drachm (ʒ).	12 ounces = 1 pound (lb.).

Measures of Value.

UNITED STATES STANDARD.

10 mills = 1 cent.	10 dimes = 1 dollar.
10 cents = 1 dime.	10 dollars = 1 eagle.

The *standard* of gold and silver is 900 parts of pure metal and 100 of alloy in 1000 parts of coin.

The *fineness* expresses the quantity of pure metal in 1000 parts.

The *remedy of the mint* is the allowance for deviation from the exact standard fineness and weight of coins.

Weight of Coin.

Double eagle	= 516	troy grains.
Eagle	= 258	troy grains.
Dollar (gold)	= 25.8	troy grains.
Dollar (silver)	= 412.5	troy grains.
Half-dollar	= 192	troy grains.
5-cent piece (nickel)	= 77.16	troy grains.
3-cent piece (nickel)	= 30	troy grains.
Cent (bronze)	= 48	troy grains.

Measure of Time.

60 seconds = 1 minute.	365 days = 1 common year.
60 minutes = 1 hour.	366 days = 1 leap year.
24 hours = 1 day.	

A *solar day* is measured by the rotation of the earth upon its axis with respect to the sun.

In *astronomical computation* and in *nautical time* the day commences at noon, and in the former it is counted throughout the 24 hours.

In *civil computation* the day commences at midnight, and is divided into two portions of 12 hours each.

A *solar year* is the time in which the earth makes one revolution around the sun; and its average time, called the *mean solar year,*

is 365 days, 5 hours, 48 minutes, 49.7 seconds, or nearly 365¼ days.

A mean lunar month, or lunation of the moon, is 29 days, 12 hours, 44 minutes, 2 seconds, and 5.24 thirds.

The Calendar, Old and New Style.

The *Julian* Calendar was established by Julius Cæsar, 44 B.C., and by it one day was inserted in every fourth year. This was the same thing as assuming that the length of the solar year was 365 days, 6 hours, instead of the value given above, thus introducing an accumulative error of 11 minutes, 12 seconds, every year. This calendar was adopted by the church in 325 A.D., at the Council of Nice. In the year 1582 the annual error of 11 minutes, 12 seconds, had amounted to a period of 10 days, which, by order of Pope Gregory XIII., was suppressed in the calendar, and the 5th of October reckoned as the 15th. To prevent the repetition of this error, it was decided to leave out three of the inserted days every 400 years, and to make this omission in the years which are not exactly divisible by 400. Thus, of the years 1700, 1800, 1900, 2000, all of which are leap years according to the Julian Calendar, only the last is a leap year according to the *Reformed* or *Gregorian* Calendar. This Reformed Calendar was not adopted by England until 1752, when 11 days were omitted from the calendar. The two calendars are now often called the *Old Style* and the *New Style*.

The latter style is now adopted in every Christian country except Russia.

Circular and Angular Measures.

USED FOR MEASURING ANGLES AND ARCS, AND FOR DETERMINING LATITUDE AND LONGITUDE.

60 seconds	(″)	= 1 minute	(′).
60 minutes		= 1 degree	(°).
360 degrees		= 1 circumference	(C.).

Seconds are usually subdivided into tenths and hundredths.

A *minute* of the circumference of the earth is a geographical mile.

Degrees of the earth's circumference on a meridian average 69.16 common miles.

THE METRIC SYSTEM.

The *metric system* is a system of weights and measures based upon a unit called a meter.

The *meter* was intended to be one ten-millionth part of the distance from the equator to either pole, measured on the earth's surface at the level of the sea.

The *names* of derived metric denominations are formed by prefixing to the name of the primary unit of measure—

Milli (mill′e), a thousandth,
Centi (sent′e), a hundredth,
Deci (des′e), a tenth,
Deka (dek′a), ten.
Hecto (hek′to), one hundred,
Kilo (kil′o), a thousand,
Myria (mir′ea), ten thousand.

This system, first adopted by France, has been extensively adopted by other countries, and is much used in the sciences and the arts. It was legalized in 1866 by Congress to be used in the United States, and is already employed by the Coast Survey, and, to some extent, by the Mint and the General Post-Office.

Linear Measures.

The *meter* is the primary unit of lengths.

TABLE.

10 millimeters (mm.)	= 1 centimeter (cm.)	= 0.3937 in.
10 centimeters	= 1 decimeter	= 3.937 in.
10 decimeters	= 1 METER	= 39.37 in.
10 meters	= 1 dekameter	= 393.37 in.
10 dekameters	= 1 hectometer	= 328 ft. 1 in.
10 hectometers	= 1 KILOMETER (km.)	= 0.62137 miles.
10 kilometers	= 1 myriameter	= 6.2137 miles.

The *meter* is used in ordinary measurements; the *centimeter*, or *millimeter*, in reckoning very small distances; and the *kilometer*, for roads or great distances.

A *centimeter* is about $\frac{3}{8}$ of an inch; a *meter* is about 3 feet 3 inches and $\frac{3}{8}$; a *kilometer* is about 200 rods, or $\frac{5}{8}$ of a mile (see p. 35).

Surface Measures.

The *square meter* is the primary unit of ordinary surfaces.

The *are* (air), a square, each of whose sides is ten *meters*, is the unit of land measures.

TABLE.

100 square millimeters (sq. mm.) = 1 square centimeter (sq. cm.)	= 0.155 sq. inch.
100 square centimeters = 1 square decimeter	= 15.5 sq. inches.
100 square decimeters = 1 square METER (sq. m.)	= 1550 sq. in., or 1.196 sq. yds.

ALSO

100 centiares, or sq. meters,	= 1 ARE (ar.)	= 119.6 sq. yds.
100 ares	= 1 hectare (ha.)	= 2.471 acres.

A *square meter*, or one *centiare*, is about $10\frac{3}{4}$ square feet, or $1\frac{1}{5}$ square yards, and a *hectare* is about $2\frac{1}{2}$ acres.

Cubic Measures.

The *cubic meter*, or *stere* (stair), is the primary unit of a volume.

TABLE.

1000 cubic millimeters (cu. mm.) = 1 cubic centimeter (cu. cm.) = [0.061 cubic inch.

1000 cubic centimeters = 1 cubic decimeter = 61.022 cubic inches.

1000 cubic decimeters = 1 cubic METER (cu. m.) = 35.314 cu. ft.

The *stere* is the name given to the cubic meter in measuring wood and timber. A tenth of a stere is a *decistere*, and ten steres are a *dekastere*.

A *cubic meter*, or *stere*, is about $1\frac{1}{3}$ cubic yards, or about $2\frac{1}{5}$ cord feet.

Liquid and Dry Measures.

The *liter* (leeter) is the primary unit of measures of capacity, and is a cube, each of whose edges is a tenth of a meter in length.

The *hectoliter* is the unit in measuring large quantities of grain, fruits, roots, and liquids.

10 milliliters (ml.)	= 1 centiliter (cl.)	= 0.338 fluid ounce.
10 centiliters	= 1 deciliter	= 0.845 liquid gill.
10 deciliters	= 1 LITER (l.)	= 1.0567 liquid quarts.
10 liters	= 1 dekaliter	= 2.6417 gallons.
10 dekaliters	= 1 HECTOLITER (hl.)	= 2 bushels 3.35 pecks.
10 hectoliters	= 1 kiloliter	= 28 bushels $1\frac{1}{2}$ pecks.

A *centiliter* is about $\frac{1}{3}$ of a fluid ounce; a *liter* is about $1\frac{1}{18}$ liquid quarts, or $\frac{9}{10}$ of a dry quart; a *hectoliter* is about $2\frac{5}{6}$ bushels; and a *kiloliter* is one cubic meter, or stere.

Weights.

The *gram* is the primary unit of weights, and is the weight in a vacuum of a cubic centimeter of distilled water at the temperature of 39.2 degrees Fahrenheit.

TABLE.

10 milligrams (mg.)	= 1 centigram	=	0.1543	troy grain.
10 centigrams	= 1 decigram	=	1.543	troy grains
10 decigrams	= 1 GRAM (g.)	=	15.432	troy grains
10 grams	= 1 dekagram	=	0.3527	avoir. oz.
10 dekagrams	= 1 hectogram	=	3.5274	avoir. oz.
10 hectograms	= 1 KILOGRAM (k.)	=	2.2046	avoir. lbs.
10 kilograms	= 1 myriagram	=	22.046	avoir. lbs.
10 myriagrams	= 1 quintal	=	220.46	avoir. lbs.
10 quintals	= 1 TONNEAU (t.)	=	2204.6	avoir. lbs.

1 kilogram per kilometer = .67195 pounds per 1000 feet.
1 pound per thousand feet = 1.4882 kilogrames per kilometer.
1 kilogram per square millimeter = 1423 pounds per square inch.
1 pound per square inch = .000743 kilograms per square [millimeter.

The *gram* is used in weighing gold, jewels, letters, and small quantities of things. The *kilogram*, or, for brevity, *kilo*, is used by grocers; and the *tonneau* (tonno), or *metric ton*, is used in finding the weight of very heavy articles.

A *gram* is about 15½ grains troy; the *kilo* about 2½ pounds avoirdupois; and the *metric ton*, about 2205 pounds.

A *kilo* is the weight of a liter of water at its greatest density; and the *metric ton*, of a cubic meter of water.

Metric numbers are written with the decimal-point (.) at the right of the figures denoting the unit; thus, 15 meters, 3 centimeters, are written, 15.03 m.

When metric numbers are expressed by figures, the part of the expression at the left of the decimal-point is read as the number of the unit, and the part at the right, if any, as a number of the lowest denomination indicated, or as a decimal part of the unit; thus, 46.525 m. is read 46 meters and 525 millimeters, or 46 and 525 thousandths meters.

In writing and reading metric numbers, according as the scale is 10, 100, or 1000, each denomination should be allowed one, two, or three orders of figures.

METRIC CONVERSION TABLE.

The following metric conversion table has been compiled by Mr. C. W. Hunt, M. Am. Soc. M. E., President of the C. W. Hunt Company, of New York City, and is most convenient in dealing with metric weights and measures:

Millimeters × .03937	= inches.
Millimeters ÷ 25.4	= inches.
Centimeters × .3937	= inches.
Centimeters ÷ 2.54	= inches.
Meters × 39.37	= ins. (Act of Congress.)
Meters × 3.281	= feet.
Meters × 1.094	= yards.
Kilometers × .621	= miles.
Kilometers ÷ 1.6093	= miles.
Kilometers × 3280.7	= feet.
Square millimeters × .0155	= square inches.
Square millimeters ÷ 645.1	= square inches.
Square centimeters × .155	= square inches.
Square centimeters ÷ 6.451	= square inches.
Square meters × 10.764	= square feet.
Square kilometers × 247.1	= acres.
Hectares × 2.471	= acres.
Cubic centimeters ÷ 16.383	= cubic inches.
Cubic centimeters ÷ 3.69	= fluid drachms. (U. S. P.)
Cubic centimeters ÷ 29.57	= fluid ounce. (U. S. P.)
Cubic meters × 35.315	= cubic feet.
Cubic meters × 1.308	= cubic yards.
Cubic meters × 264.2	= gallons (231 cu. ins.).
Liters × 61.022	= cu ins. (Act of Congress.)
Liters × 33.84	= fluid ounces. (U. S. P.
Liters × .2642	= gallons (231 cu. ins.).
Liters ÷ 3.78	= gallons (231 cu. ins.).
Liters ÷ 28.316	= cubic feet.
Hectoliters × 3.531	= cubic feet.
Hectoliters × 2.84	= bushels (2150.42 cu. ins.).
Hectoliters × .131	= cubic yards.
Hectoliters ÷ 26.42	= gallons (231 cu. ins.).
Grammes × 15.432	= grains. (Act of Congress.)
Grammes × 981	= dynes.
Grammes (water) ÷ 29.57	= fluid ounces.
Grammes ÷ 28.35	= ounces avoirdupois.
Grammes per cu. cent. ÷ 27.7	= pounds per cubic inch.
Joule × .7373	= foot-pounds.
Kilograms × 2.2046	= pounds.
Kilogrammes × 35.3	= ounces avoirdupois.
Kilograms ÷ 1102.3	= tons (2000 pounds).
Kilograms per sq. cent . × 14.223	= pounds per square inch.

Kilogrammeters × 7.233	= foot-pounds.
Kilograms per meter × .672	= pounds per square foot.
Kilograms per cubic meter × .062	= pounds per cubic foot.
Kilograms per cheval vapeur × 2.235	= pounds per horse-power.
Kilo-watts × 1.34	= horse-power.
Watts ÷ 746	= horse-power.
Watts ÷ .7373	= foot-pounds per second.
Calorie × 3.968	= B. T. U.
Cheval vapeur × .9863	= horse-power.
(Centigrade × 1.8) + 32	= degrees Fahrenheit.
Francs × .193	= dollars.
Gravity, Paris	= 980.94 cent. per second.

SCRIPTURE AND ANCIENT MEASURES AND WEIGHTS.

Scripture Long Measures.

	Inches.		*Feet.*	*Inches.*
Digit	= 0.912	Cubit	= 1	9.888
Palm	= 3.648	Fathom	= 7	3.552
Span	= 10.944			

Egyptian Long Measures.

Nahud cubit = 1 foot 5.71 ins. Royal cubit = 1 foot 8.66 ins.

Grecian Long Measure.

	Feet.	*Inches.*		*Feet.*	*Inches.*
Digit	=	0.7554	Stadium	= 604	4.5
Pous (foot)	= 1	0.0875	Mile	= 4835	
Cubit	= 1	1.5984⅜			

Jewish Long Measures.

Cubit	= 1.824 ft.	Mile	= 7296 feet.
Sabbath-day's journey	= 3648 ft.	Day's journey	= 33.164 miles.

Roman Long Measures.

	Inches.		*Feet.*	*Inches.*
Digit	= 0.72575	Cubit	= 1	5.406
Uncia (inch)	= 0.967	Passus	= 4	10.02
Pes (foot)	= 11.604	Mile (millarium)	= 4842	

Roman Weight.

Ancient libbra = 0.7094 pound.

Miscellaneous.

	Feet.		*Feet.*
Arabian foot	= 1.095	Hebrew foot	= 1.212
Babylonian foot	= 1.140	Hebrew cubit	= 1.817
Egyptian finger	= 0.06145	Hebrew sacred cubit	= 2.002

Metrical Conversion Tables.—This and the following table from "Molesworth's Metrical Tables" will be found of great convenience in figuring plans to be executed in Mexico and other countries using the metric system.

FEET CONVERTED INTO METRES.

Feet.	0	1	2	3	4	5	6	7	8	9	Feet.
0	....	.304794	.609589	.914383	1.21918	1.52397	1.82877	2.13356	2.43836	2.74315	0
10	3.047945	3.35274	3.65753	3.96233	4.26712	4.57192	4.87671	5.18151	5.48630	5.79110	10
20	6.095890	6.40068	6.70548	7.01027	7.31507	7.61986	7.92466	8.22945	8.53425	8.83904	20
30	9.143835	9.44863	9.75342	10.0582	10.3630	10.6678	10.9726	11.2774	11.5822	11.8870	30
40	12.19178	12.4966	12.8014	13.1062	13.4110	13.7158	14.0205	14.3253	14.6301	14.9349	40
50	15.23972	15.5445	15.8493	16.1541	16.4589	16.7637	17.0685	17.3733	17.6781	17.9829	50
60	18.28767	18.5925	18.8973	19.2020	19.5068	19.8116	20.1164	20.4212	20.7260	21.0308	60
70	21.33561	21.6404	21.9452	22.2500	22.5548	22.8596	23.1644	23.4692	23.7740	24.0788	70
80	24.38356	24.6884	24.9931	25.2979	25.6027	25.9075	26.2123	26.5171	26.8219	27.1267	80
90	27.43150	27.7363	28.0411	28.3459	28.6507	28.9555	29.2603	29.5651	29.8699	30.1747	90
Feet.	0	1	2	3	4	5	6	7	8	9	Feet.

Example.—44 feet = 13.411 metres, = 134.11 decimetres, = 1341.1 centimeters, = 13411 millimetres.

The above-mentioned work contains eighty pages of conversion tables similar to the above. Published by Spon & Chamberlain, 123 Liberty St., N. Y.

INCHES AND SIXTEENTHS CONVERTED INTO MILLIMETRES.

Ins.	0	1	2	3	4	5	6	7	8	9	10	11	Ins.
	....	25.400	50.799	76.199	101.60	127.00	152.40	177.80	203.20	228.60	254.00	279.39	
1/16	1.5875	26.987	52.387	77.786	103.19	128.59	153.98	179.38	204.78	230.18	255.58	280.98	1/16
1/8	3.1749	28.574	53.974	79.374	104.77	130.17	155.57	180.97	206.37	231.77	257.17	282.57	1/8
3/16	4.7624	30.162	55.561	80.961	106.36	131.76	157.16	182.56	207.96	233.36	258.76	284.16	3/16
1/4	6.3499	31.749	57.149	82.549	107.95	133.35	158.75	184.15	209.55	234.95	260.35	285.74	1/4
5/16	7.9374	33.337	58.736	84.136	109.54	134.94	160.33	185.73	211.13	236.53	261.93	287.33	5/16
3/8	9.5248	34.924	60.324	85.723	111.12	136.52	161.92	187.32	212.72	238.12	263.52	288.92	3/8
7/16	11.112	66.512	61.911	87.311	112.71	138.11	163.51	188.91	214.31	239.71	265.11	290.51	7/16
1/2	12.700	38.099	63.499	88.898	114.30	139.70	165.10	190.50	215.90	241.30	266.70	292.09	1/2
9/16	14.287	39.687	65.086	90.486	115.89	141.28	166.68	192.08	217.48	242.88	268.28	293.68	9/16
5/8	15.875	41.274	66.674	92.073	117.47	142.87	168.27	193.67	219.07	244.47	269.87	295.27	5/8
11/16	17.462	42.862	68.261	93.661	119.06	144.46	169.86	195.26	220.66	246.06	271.46	296.86	11/16
3/4	19.050	44.449	69.849	95.248	120.65	146.05	171.45	196.85	222.25	247.65	273.05	298.44	3/4
13/16	20.637	46.037	71.436	96.836	122.24	147.63	173.03	198.43	223.83	249.23	274.63	300.03	13/16
7/8	22.225	47.624	73.024	98.423	123.82	149.22	174.62	200.02	225.42	250.82	276.22	301.62	7/8
15/16	23.812	49.212	74.611	100.01	125.41	150.81	176.21	201.61	227.01	252.41	277.81	303.21	15/16
Ins.	0	1	2	3	4	5	6	7	8	9	10	11	Ins.

For metres move the decimal point *three* figures forward.

EXAMPLE.—8 3/16 = 207.96 millimetres, = 20.796 centimetres, = 2.0796 decimetres, = .20796 metre.

MENSURATION.

Definitions.

A *point* is that which has only position.

A *plane* is a surface in which, any two points being taken, the straight line joining them will be wholly in the surface.

Fig. 1

A Curved Line.

A *curved line* is a line of which no portion is straight (Fig. 1).

Parallel lines are such as are wholly in the same plane, and have the same direction (Fig. 2).

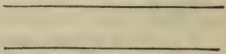


Fig. 2

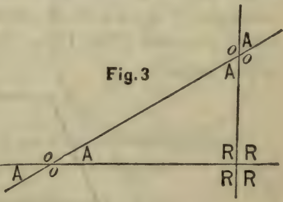
Parallel Lines.

A *broken line* is a line composed of a series of dashes; thus, —— —— ——.

An *angle* is the opening between two lines meeting at a point, and is termed a *right angle* when the two lines are perpendicular to each other, an *acute angle* when it is less or sharper than a right angle, and *obtuse* when it is greater than a right angle. Thus, in Fig. 3,

A A A A are *acute angles*,

o o o o are *obtuse angles*, R R R R are *right angles*.



Polygons.

A *polygon* is a portion of a plane bounded by straight lines.

A *triangle* is a polygon of three sides.

A *scalene triangle* has none of its sides equal; an *isosceles triangle* has two of its sides equal; an *equilateral triangle* has all three of its sides equal.

A *right-angled triangle* is one which has a right angle. The side opposite the right angle is called the *hypothenuse;* the side on which the triangle is supposed to stand is called its *base*, and the other side, its *altitude*.

Fig. 4.

Fig. 5.
Scalene Triangle.

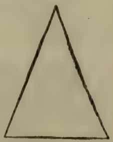


Fig. 6.
Isosceles Triangle.

Fig. 7.
Equilateral Triangle.

A *quadrilateral* is a polygon of four sides.

Quadrilaterals are divided into classes, as follows,—the *trapezium* (Fig. 8), which has no two of its sides parallel; the *trapezoid* (Fig. 9), which has two of its sides parallel; and the *parallelogram* (Fig. 10), which is bounded by two pairs of parallel sides.


Fig. 8.


Fig. 9.


Fig. 10.

A parallelogram whose sides are not equal, and its angles not right angles, is called a *rhomboid* (Fig. 11); when the sides are all equal, but the angles are not right angles, it is called a *rhombus* (Fig. 12); and, when the angles are right angles, it is called a *rectangle* (Fig. 13). A rectangle whose sides are all equal is called a *square* (Fig. 14). Polygons whose sides are all equal are called *regular*.


Fig. 11.


Fig. 12.


Fig. 13.


Fig. 14.

Besides the square and equilateral triangles, there are
The *pentagon* (Fig. 15), which has five sides;
The *hexagon* (Fig. 16), which has six sides;
The *heptagon* (Fig. 17), which has seven sides;
The *octagon* (Fig. 18), which has eight sides.


Fig. 15.


Fig. 16.


Fig. 17.

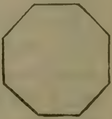


Fig. 18.

The *enneagon* has nine sides.
The *decagon* has ten sides.
The *dodecagon* has twelve sides.

For all polygons, the side upon which it is supposed to stand is called its *base;* the perpendicular distance from the highest side or angle to the base (prolonged, if necessary) is called the *altitude;* and a line joining any two angles not adjacent is called a *diagonal.*

A *perimeter* is the boundary line of a plane figure.

A *circle* is a portion of a plane bounded by a curve, all the points of which are equally distant from a point within called the centre (Fig. 19).

The *circumference* is the curve which bounds the circle.

A *radius* is any straight line drawn from the centre to the circumference.

Any straight line drawn through the centre to the circumference on each side is called a *diameter*.

An *arc* of a circle is any part of its circumference.

A *chord* is any straight line joining two points of the circumference, as *bd*.

A *segment* is a portion of the circle included between the arc and its chord, as A in Fig. 19.

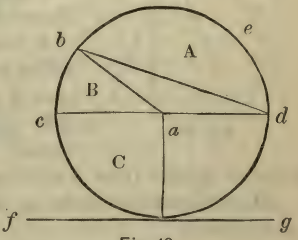


Fig. 19.

A *sector* is the space included between an arc and two radii drawn to its extremities, as B, Fig. 19. In the figure, *ab* is a radius, *cd* a diameter, and *db* is a chord *subtending* the arc *bed*. A *tangent* is a right line which in passing a curve touches without cutting it, as *fg*, Fig. 19.

Volumes.

A *prism* is a volume whose ends are equal and parallel polygons, and whose sides are parallelograms.

A prism is *triangular*, *rectangular*, etc., according as its ends are *triangles*, *rectangles*, etc.

A *cube* is a rectangular prism all of whose sides are squares.

A *cylinder* is a volume of uniform diameter, bounded by a curved surface and two equal and parallel circles.

A *pyramid* is a volume whose base is a polygon, and whose sides are triangles meeting in a point called the *vertex*.

A pyramid is triangular, quadrangular, etc., according as its base is a triangle, quadrilateral, etc.

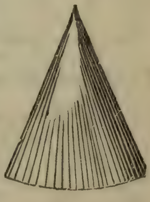


Fig. 20.

A *cone* is a volume whose base is a circle, from which the remaining surface tapers uniformly to a point or vertex (Fig. 20).

Conic sections are the figures made by a plane cutting a cone.

An *ellipse* is the section of a cone when cut by a plane passing obliquely through both sides, as at *ab*, Fig. 21.

A *parabola* is a section of a cone cut by a plane parallel to its side, as at *cd*.

A *hyperbola* is a section of a cone cut by a plane at a greater angle through the base than is made by the side of the cone, as at *eh*.

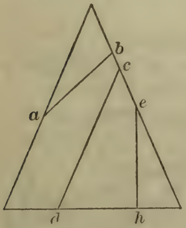


Fig. 21.

In the ellipse, the *transverse axis*, or *long diameter*, is the longest line that can be drawn through it. The *conjugate axis*, or *short diameter*, is a line drawn through the centre, at right angles to the long diameter.

A *frustum of a pyramid* or *cone* is that which remains after cutting off the upper part of it by a plane parallel to the base.

A *sphere* is a volume bounded by a curved surface, all points of which are equally distant from a point within, called the centre.

Mensuration *treats of the measurement of lines, surfaces, and volumes.*

RULES.

To compute the area of a square, a rectangle, a rhombus, or a rhomboid.

Rule.—Multiply the length by the breadth or height; thus, in either of Figs. 22, 23, 24, the area $= ab \times bc$.

Fig.22

Fig.23

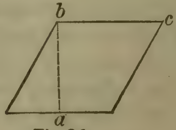


Fig.24

To compute the area of a triangle.

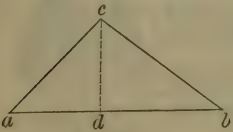


Fig.25

Rule.—Multiply the base by the altitude, and divide by 2; thus, in Fig. 25, the area of $abc = \dfrac{ab \times cd}{2}$.

To find the length of the hypothenuse of a right-angled triangle when both sides are known.

RULE.—Square the length of each of the sides making the right angle, add their squares together, and take the square root of their sum. Thus (Fig. 26), the length of $ac=3$, and of $bc=4$; then

$$ab=3\times3=9+(4\times4)=9+16=25.$$
$$\sqrt{25}=5, \quad \text{or} \quad ab=5.$$

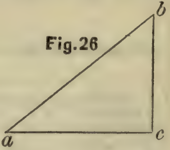


Fig.26

To find the length of the base or altitude of a right-angled triangle when the length of the hypothenuse and one side is known.

RULE.—From the square of the length of the hypothenuse subtract the square of the length of the other side, and take the square root of the remainder.

To find the area of a trapezium.

RULE.—Multiply the diagonal by the sum of the two perpendiculars falling upon it from the opposite angles, and divide the product by 2. Or,

$$\frac{ab\times(ce+di)}{2}=\text{area (Fig. 27).}$$

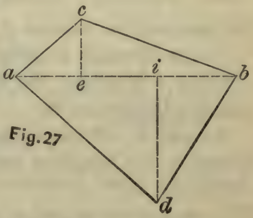


Fig.27

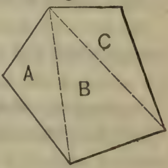
To find the area of a trapezoid (Fig. 28).

RULE.—Multiply the sum of the two parallel sides by the perpendicular distance between them, and divide the product by 2.


Fig. 28

To compute the area of an irregular polygon.

RULE.—Divide the polygon into triangles by means of diagonal lines, and then add together the areas of all the triangles, as *A*, *B*, and *C* (Fig. 29).

Fig.29


To find the area of a regular polygon.

RULE.—Multiply the length of a side by the perpendicular distance to the centre (as *ao*, Fig. 30), and that product by the numbe of sides, and divide the result by 2.

To compute the area of a regular polygon when the length of a side only is given.

RULE.—Multiply the square of the side by the multiplier opposite the name of the polygon in column *A* of the following table:

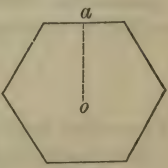


Fig.30

Name of Polygon.	No. of sides.	A. Area.	B. Radius of circumscribing circle.	C. Length of the sides.	D. Radius of inscribed circle.
Triangle.	3	0.433013	0.5773	1.732	0.2887
Tetragon	4	1	0.7071	1.4142	0.5
Pentagon.	5	1.720477	0.8506	1.1756	0.6882
Hexagon.	6	2.598076	1	1	0.866
Heptagon.	7	3.633912	1.1524	0.8677	1.0383
Octagon.	8	4.828427	1.3066	0.7653	1.2071
Nonagon.	9	6.181824	1.4619	0.684	1.3737
Decagon.	10	7.694209	1.618	0.618	1.5383
Undecagon.	11	9.36564	1.7747	0.5634	1.7028
Dodecagon.	12	11.196152	1.9319	0.5176	1.866

To compute the radius of a circumscribing circle when the length of a side only is given.

Rule.—Multiply the length of a side of the polygon by the number in column *B*.

Example.—What is the radius of a circle that will contain a hexagon, the length of one side being 5 inches?

Ans. $5 \times 1 = 5$ inches.

To compute the length of a side of a polygon that is contained in a given circle, when the radius of the circle is given.

Rule.—Multiply the radius of the circle by the number opposite the name of the polygon in column *C*.

Example.—What is the length of the side of a pentagon contained in a circle 8 feet in diameter?

Ans. 8 ft. diameter $\div 2 = 4$ ft. radius, $4 \times 1.1756 = 4.7024$ ft.

To compute the length of a side of a regular polygon, when the radius of the inscribed circle is given.

Rule.—Divide the radius of the inscribed circle by the number opposite the name of the polygon in column *D*.

To compute the radius of a circle that can be inscribed in a given polygon, when the length of a side is given.

Rule.—Multiply the length of a side of the polygon by the number opposite the name of the polygon in column *D*.

Example.—What is the radius of the circle that can be inscribed in an octagon, the length of one side being 6 inches.

Ans. $6 \times 1.2071 = 7.2426$ inches.

Circles.

To compute the circumference of a circle.

Rule.—Multiply the diameter by 3.1416; or, for most purposes, by $3\frac{1}{7}$ is sufficiently accurate.

EXAMPLE.—What is the circumference of a circle 7 inches in diameter?

Ans. $7 \times 3.1416 = 21.9912$ inches, or $7 \times 3\frac{1}{7} = 22$ inches, the error in this last being 0.0088 of an inch.

To find the diameter of a circle when the circumference is given.

RULE.—Divide the circumference by 3.1416, or for a very close approximate result multiply by 7 and divide by 22.

To find the radius of an arc, when the chord and rise or versed sine are given.

RULE.—Square *one-half* the *chord*, also square the *rise;* divide their sum by twice the rise; the result will be the radius.

EXAMPLE.—The length of the chord *ac*, Fig. 30½, is 48 inches, and the rise, *bo*, is 6 inches. What is the radius of the arc?

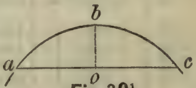


Fig. 30½

Ans. $\text{Rad} = \frac{oc^2 + bo^2}{2bo} = \frac{24^2 + 6^2}{12} = 51$ ins.

To find the rise or versed sine of a circular arc, when the chord and radius are given.

RULE.—Square the radius; also square one-half the chord; subtract the latter from the former, and take the square root of the remainder. Subtract the result from the radius, and the remainder will be the rise.

EXAMPLE.—A given arc has a radius of 51 inches, and a chord of 48 inches. What is the rise?

Ans. $\text{Rise} = \text{rad} - \sqrt{\text{rad}^2 - \tfrac{1}{2}\text{chord}^2} = 51 - \sqrt{2601 - 576}$
$= 51 - 45 = 6$ inches $=$ rise.

To compute the area of a circle.

RULE.—Multiply the square of the diameter by 0.7854, or multiply the square of the radius by 3.1416.

EXAMPLE.—What is the area of a circle 10 inches in diameter?

Ans. $10 \times 10 \times 0.7854 = 78.54$ square inches, or $5 \times 5 \times 3.1416 = 78.54$ square inches.

The following tables will be found very convenient for finding the circumference and area of circles.

AREAS AND CIRCUMFERENCES OF CIRCLES.

(For Diameters from $\frac{1}{10}$ to 100, advancing by Tenths.)

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
0.0			5.0	19.6350	15.7080	10.0	78.5398	31.4159
.1	0.007854	0.31416	.1	20.4282	16.0221	.1	80.1185	31.7301
.2	0.031416	0.62832	.2	21.2372	16.3363	.2	81.7128	32.0442
.3	0.070686	0.94248	.3	22.0618	16.6504	.3	83.3229	32.3584
.4	0.12566	1.2566	.4	22.9022	16.9646	.4	84.9487	32.6726
.5	0.19635	1.5708	.5	23.7583	17.2788	.5	86.5901	32.9867
.6	0.28274	1.8850	.6	24.6301	17.5929	.6	88.2473	33.3009
.7	0.38485	2.1991	.7	25.5176	17.9071	.7	89.9202	33.6150
.8	0.50266	2.5133	.8	26.4208	18.2212	.8	91.6088	33.9292
.9	0.63617	2.8274	.9	27.3397	18.5354	.9	93.3132	34.2434
1.0	0.7854	3.1416	6.0	28.2743	18.8496	11.0	95.0332	34.5575
.1	0.9503	3.4558	.1	29.2247	19.1637	.1	96.7689	34.8717
.2	1.1310	3.7699	.2	30.1907	19.4779	.2	98.5203	35.1858
.3	1.3273	4.0841	.3	31.1725	19.7920	.3	100.2875	35.5000
.4	1.5394	4.3982	.4	32.1699	20.1062	.4	102.0703	35.8142
.5	1.7671	4.7124	.5	33.1831	20.4204	.5	103.8689	36.1283
.6	2.0106	5.0265	.6	34.2119	20.7345	.6	105.6832	36.4425
.7	2.2698	5.3407	.7	35.2565	21.0487	.7	107.5132	36.7566
.8	2.5447	5.6549	.8	36.3168	21.3628	.8	109.3588	37.0708
.9	2.8353	5.9690	.9	37.3928	21.6770	.9	111.2202	37.3850
2.0	3.1416	6.2832	7.0	38.4845	21.9911	12.0	113.0973	37.6991
.1	3.4636	6.5973	.1	39.5919	22.3053	.1	114.9901	38.0133
.2	3.8013	6.9115	.2	40.7150	22.6195	.2	116.8987	38.3274
.3	4.1548	7.2257	.3	41.8539	22.9336	.3	118.8229	38.6416
.4	4.5239	7.5398	.4	43.0084	23.2478	.4	120.7628	38.9557
.5	4.9087	7.8540	.5	44.1786	23.5619	.5	122.7185	39.2699
.6	5.3093	8.1681	.6	45.3646	23.8761	.6	124.6898	39.5841
.7	5.7256	8.4823	.7	46.5663	24.1903	.7	126.6769	39.8982
.8	6.1575	8.7965	.8	47.7836	24.5044	.8	128.6796	40.2124
.9	6.6052	9.1106	.9	49.0167	24.8186	.9	130.6981	40.5265
3.0	7.0686	9.4248	8.0	50.2655	25.1327	13.0	132.7323	40.8407
.1	7.5477	9.7389	.1	51.5300	25.4469	.1	134.7822	41.1549
.2	8.0425	10.0531	.2	52.8102	25.7611	.2	136.8478	41.4690
.3	8.5530	10.3673	.3	54.1061	26.0752	.3	138.9291	41.7832
.4	9.0792	10.6814	.4	55.4177	26.3894	.4	141.0261	42.0973
.5	9.6211	10.9956	.5	56.7450	26.7035	.5	143.1388	42.4115
.6	10.1788	11.3097	.6	58.0880	27.0177	.6	145.2672	42.7257
.7	10.7521	11.6239	.7	59.4468	27.3319	.7	147.4114	43.0398
.8	11.3411	11.9381	.8	60.8212	27.6460	.8	149.5712	43.3540
.9	11.9459	12.2522	.9	62.2114	27.9602	.9	151.7468	43.6681
4.0	12.5664	12.5664	9.0	63.6173	28.2743	14.0	153.9380	43.9823
.1	13.2025	12.8805	.1	65.0388	28.5885	.1	156.1450	44.2965
.2	13.8544	13.1947	.2	66.4761	28.9027	.2	158.3677	44.6106
.3	14.5220	13.5088	.3	67.9291	29.2168	.3	160.6061	44.9248
.4	15.2053	13.8230	.4	69.3978	29.5310	.4	162.8602	45.2389
.5	15.9043	14.1372	.5	70.8822	29.8451	.5	165.1300	45.5531
.6	16.6190	14.4513	.6	72.3823	30.1593	.6	167.4155	45.8673
.7	17.3494	14.7655	.7	73.8981	30.4734	.7	169.7167	46.1814
.8	18.0956	15.0796	.8	75.4296	30.7876	.8	172.0336	46.4956
.9	18.8574	15.3938	.9	76.9769	31.1018	.9	174.3662	46.8097

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
15.0	176.7146	47.1239	20.0	314.1593	62.8319	25.0	490.8739	78.5398
.1	179.0786	47.4380	.1	317.3087	63.1460	.1	494.8087	78.8540
.2	181.4584	47.7522	.2	320.4739	63.4602	.2	498.7592	79.1681
.3	183.8539	48.0664	.3	323.6547	63.7743	.3	502.7255	79.4823
.4	186.2650	48.3805	.4	326.8513	64.0885	.4	506.7075	79.7965
.5	188.6919	48.6947	.5	330.0636	64.4026	.5	510.7052	80.1106
.6	191.1345	49.0088	.6	333.2916	64.7168	.6	514.7185	80.4248
.7	193.5928	49.3230	.7	336.5353	65.0310	.7	518.7476	80.7389
.8	196.0668	49.6372	.8	339.7947	65.3451	.8	522.7924	81.0531
.9	198.5565	49.9513	.9	343.0698	65.6593	.9	526.8529	81.3672
16.0	201.0619	50.2655	21.0	346.3606	65.9734	26.0	530.9292	81.6814
.1	203.5831	50.5796	.1	349.6671	66.2876	.1	535.0211	81.9956
.2	206.1199	50.8938	.2	352.9894	66.6018	.2	539.1287	82.3097
.3	208.6724	51.2080	.3	356.3273	66.9159	.3	543.2521	82.6239
.4	211.2407	51.5221	.4	359.6809	67,2301	.4	547.3911	82.9380
.5	213.8246	51.8363	.5	363.0503	67.5442	.5	551.5459	83.2522
.6	216.4243	52.1504	.6	366.4354	67.8584	.6	555.7163	83.5664
.7	219.0397	52.4646	.7	369.8361	68.1726	.7	559.9025	83.8805
.8	221.6708	52.7788	.8	373.2526	68.4867	.8	564.1044	84.1947
.9	224.3176	53.0929	.9	376.6848	68.8009	.9	568.3220	84.5088
17.0	226.9801	53.4071	22.0	380.1327	69.1150	27.0	572.5553	84.8230
.1	229.6583	53.7212	.1	383.5963	69.4292	.1	576.8043	85.1372
.2	232.3522	54.0354	.2	387.0756	69.7434	.2	581.0690	85.4513
.3	235.0618	54.3496	.3	390.5707	70.0575	.3	585.3494	85.7655
.4	237.7871	54.6637	.4	394.0814	70.3717	.4	589.6455	86.0796
.5	240.5282	54.9779	.5	397.6078	70.6858	.5	593.9574	86.3938
.6	243.2849	55.2920	.6	401.1500	71.0000	.6	598.2849	86.7080
.7	246.0574	55.6062	.7	404.7078	71.3142	.7	602.6282	87.0221
.8	248.8456	55.9203	.8	408.2814	71.6283	.8	606.9871	87.3363
.9	251.6494	56.2345	.9	411.8707	71.9425	.9	611.3618	87.6504
18.0	254.4690	56.5486	23.0	415.4756	72.2566	28.0	615.7522	87.9646
.1	257.3043	56.8628	.1	419.0963	72.5708	.1	620.1582	88.2788
.2	260.1553	57.1770	.2	422.7327	72,8849	.2	624.5800	88.5929
.3	263.0220	57.4911	.3	426.3848	73.1991	.3	629.0175	88.9071
.4	265.9044	57.8053	.4	430.0526	73.5133	.4	633.4707	89.2212
.5	268.8025	58.1195	.5	433.7361	73.8274	.5	637.9397	89.5354
.6	271.7164	58.4336	.6	437.4354	74.1416	.6	642.4243	89.8495
.7	274.6459	58.7478	.7	441.1503	74.4557	.7	646.9246	90.1637
.8	277.5911	59.0619	.8	444.8809	74.7699	.8	651.4407	90.4779
.9	280.5521	59.3761	.9	448.6273	75.0841	.9	655.9724	90.7920
19.0	283.5287	59.6903	24.0	452.3893	75.3982	29.0	660.5199	91.1062
.1	286.5211	60.0044	.1	456.1671	75.7124	.1	665.0830	91.4203
.2	289.5292	60.3186	.2	459.9606	76.0265	.2	669,6619	91.7345
.3	292.5530	60.6327	.3	463.7698	76.3407	.3	674.2565	92.0487
.4	295.5925	60.9469	.4	467,5947	76.6549	.4	678.8668	92.3628
.5	298.6477	61.2611	.5	471.4352	76.9690	.5	683.4928	92.6770
.6	301.7186	61.5752	.6	475.2916	77.2832	.6	688.1345	92.9911
.7	304.8052	61.8894	.7	479.1636	77.5973	.7	692.7919	93.3053
.8	307.9075	62.2035	.8	483.0513	77.9115	.8	697.4650	93.6195
.9	311.0255	62.5177	.9	486.9547	78.2257	.9	702.1538	93.9336

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
30.0	706.8583	94.2478	35.0	962.1128	109.9557	40.0	1256.6371	125.6637
.1	711.5786	94.5619	.1	967.6184	110.2699	.1	1262.9281	125.9779
.2	716.3145	94.8761	.2	973.1397	110.5841	.2	1269.2348	126.2920
.3	721.0662	95.1903	.3	978.6768	110.8982	.3	1275.5573	126.6062
.4	725.8336	95.5044	.4	984.2296	111.2124	.4	1281.8955	126.9203
.5	730.6167	95.8186	.5	989.7980	111.5265	.5	1288.2493	127.2345
.6	735.4154	96.1327	.6	995.3822	111.8407	.6	1294.6189	127.5487
.7	740.2299	96.4469	.7	1000.9821	112.1549	.7	1301.0042	127.8628
.8	745.0601	96.7611	.8	1006.5977	112.4690	.8	1307.4052	128.1770
.9	749.9060	97.0752	.9	1012.2290	112.7832	.9	1313.8219	128.4911
31.0	754.7676	97.3894	36.0	1017.8760	113.0973	41.0	1320.2543	128.8053
.1	759.6450	97.7035	.1	1023.5387	113.4115	.1	1326.7024	129.1195
.2	764.5380	98.0177	.2	1029.2172	113.7257	.2	1333.1663	129.4336
.3	769.4467	98.3319	.3	1034.9113	114.0398	.3	1339.6458	129.7478
.4	774.3712	98.6460	.4	1040.6212	114.3540	.4	1346.1410	130.0619
.5	779.3113	98.9602	.5	1046.3467	114.6681	.5	1352.6520	130.3761
.6	784.2672	99.2743	.6	1052.0880	114.9823	.6	1359.1786	130.6903
.7	789.2388	99.5885	.7	1057.8449	115.2965	.7	1365.7210	131.0044
.8	794.2260	99.9026	.8	1063.6176	115.6106	.8	1372.2791	131.3186
.9	799.2290	100.2168	.9	1069.4060	115.9248	.9	1378.8529	131.6327
32.0	804.2477	100.5310	37.0	1075.2101	116.2389	42.0	1385.4424	131.9469
.1	809.2821	100.8451	.1	1081.0299	116.5531	.1	1392.0476	132.2611
.2	814.3322	101.1593	.2	1086.8654	116.8672	.2	1398.6685	132.5752
.3	819.3980	101.4734	.3	1092.7166	117.1814	.3	1405.3051	132.8894
.4	824.4796	101.7876	.4	1098.5835	117.4956	.4	1411.9574	133.2035
.5	829.5768	102.1018	.5	1104.4662	117.8097	.5	1418.6254	133.5177
.6	834.6898	102.4159	.6	1110.3645	118.1239	.6	1425.3092	133.8318
.7	839.8185	102.7301	.7	1116.2786	118.4380	.7	1432.0086	134.1460
.8	844.9628	103.0442	.8	1122.2083	118.7522	.8	1438.7238	134.4602
.9	850.1229	103.3584	.9	1128.1538	119.0664	.9	1445.4546	134.7743
33.0	855.2986	103.6726	38.0	1134.1149	119.3805	43.0	1452.2012	135.0885
.1	860.4902	103.9867	.1	1140.0918	119.6947	.1	1458.9635	135.4026
.2	865.6973	104.3009	.2	1146.0844	120.0088	.2	1465.7415	135.7168
.3	870.9202	104.6150	.3	1152.0927	120.3230	.3	1472.5352	136.0310
.4	876.1588	104.9292	.4	1158.1167	120.6372	.4	1479.3446	136.3451
.5	881.4131	105.2434	.5	1164.1564	120.9513	.5	1486.1697	136.6593
.6	886.6831	105.5575	.6	1170.2118	121.2655	.6	1493.0105	136.9734
.7	891.9688	105.8717	.7	1176.2830	121.5796	.7	1499.8670	137.2876
.8	897.2703	106.1858	.8	1182.3698	121.8938	.8	1506.7393	137.6018
.9	902.5874	106.5000	.9	1188.4724	122.2080	.9	1513.6272	137.9159
34.0	907.9203	106.8142	39.0	1194.5906	122.5221	44.0	1520.5308	138.2301
.1	913.2688	107.1283	.1	1200.7246	122.8363	.1	1527.4502	138.5442
.2	918.6331	107.4425	.2	1206.8742	123.1504	.2	1534.3853	138.8584
.3	924.0131	107.7566	.3	1213.0396	123.4646	.3	1541.3360	139.1726
.4	929.4088	108.0708	.4	1219.2207	123.7788	.4	1548.3025	139.4867
.5	934.8202	108.3849	.5	1225.4175	124.0929	.5	155 5.247	139.8009
.6	940.2473	108.6991	.6	1231.6300	124.4071	.6	1562.2826	140.1153
.7	945.6901	109.0133	.7	1237.8582	124.7212	.7	1569.2962	140.4292
.8	951.1486	109.3274	.8	1244.1021	125.0354	.8	1576.3255	140.7434
.9	956.6228	109.6416	.9	1250.3617	125.3495	.9	1583.3706	141.0575

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
45.0	1590.4313	141.3717	50.0	1963.4954	157.0796	55.0	2375.8294	172.7876
.1	1597.5077	141.6858	.1	1971.3572	157.3938	.1	2384.4767	173.1017
.2	1604.5999	142.0000	.2	1979.2348	157.7080	.2	2393.1396	173.4159
.3	1611.7077	142.3142	.3	1987.1280	158.0221	.3	2401.8183	173.7301
.4	1618.8313	142.6283	.4	1995.0370	158.3363	.4	2410.5126	174.0442
.5	1625.9705	142.9425	.5	2002.9617	158.6504	.5	2419.2227	174.3584
.6	1633.1255	143.2566	.6	2010.9020	158.9646	.6	2427.9485	174.6726
.7	1640.2962	143.5708	.7	2018.8581	159.2787	.7	2436.6899	174.9867
.8	1647.4826	143.8849	.8	2026.8299	159.5929	.8	2445.4471	175.3009
.9	1654.6847	144.1991	.9	2034.8174	159.9071	.9	2454.2200	175.6150
46.0	1661.9025	144.5133	51.0	2042.8206	160.2212	56.0	2463.0086	175.9292
.1	1669.1360	144.8274	.1	2050.8395	160.5354	.1	2471.8130	176.2433
.2	1676.3853	145.1416	.2	2058.8742	160.8495	.2	2480.6330	176.5575
.3	1683.6502	145.4557	.3	2066.9245	161.1637	.3	2489.4687	176.8717
.4	1690.9308	145.7699	.4	2074.9905	161.4779	.4	2498.3201	177.1858
.5	1698.2272	146.0841	.5	2083.0723	161.7920	.5	2507.1873	177.5000
.6	1705.5392	146.3982	.6	2091.1697	162.1062	.6	2516.0701	177.8141
.7	1712.8670	146.7124	.7	2099.2829	162.4203	.7	2524.9687	178.1283
.8	1720.2105	147.0265	.8	2107.4118	162.7345	.8	2533.8830	178.4425
.9	1727.5697	147.3407	.9	2115.5563	163.0487	.9	2542.8129	178.7566
47.0	1734.9445	147.6550	52.0	2123.7166	163.3628	57.0	2551.7586	179.0708
.1	1742.3351	147.9690	.1	2131.8926	163.6770	.1	2560.7200	179.3849
.2	1749.7414	148.2832	.2	2140.0843	163.9911	.2	2569.6971	179.6991
.3	1757.1635	148.5973	.3	2148.2917	164.3053	.3	2578.6899	180.0133
.4	1764.6012	148.9115	.4	2156.5149	164.6195	.4	2587.6985	180.3274
.5	1772.0546	149.2257	.5	2164.7537	164.9336	.5	2596.7227	180.6416
.6	1779.5237	149.5398	.6	2173.0082	165.2479	.6	2605.7626	180.9557
.7	1787.0086	149.8540	.7	2181.2785	165.5619	.7	2614.8183	181.2699
.8	1794.5091	150.1681	.8	2189.5644	165.8761	.8	2623.8896	181.5841
.9	1802.0254	150.4823	.9	2197.8661	166.1903	.9	2632.9767	181.8982
48.0	1809.5574	150.7964	53.0	2206.1834	166.5044	58.0	2642.0794	182.2124
.1	1817.1050	151.1106	.1	2214.5165	166.8186	.1	2651.1979	182.5265
.2	1824.6684	151.4248	.2	2222.8653	167.1327	.2	2660.3321	182.8407
.3	1832.2475	151.7389	.3	2231.2298	167.4469	.3	2669.4820	183.1549
.4	1839.8423	152.0531	.4	2239.6100	167.7610	.4	2678.6476	183.4690
.5	1847.4528	152.3672	.5	2248.0059	168.0752	.5	2687.8289	183.7832
.6	1855.0790	152.6814	.6	2256.4175	168.3894	.6	2697.0259	184.0973
.7	1862.7210	152.9956	.7	2264.8448	168.7035	.7	2706.2386	184.4115
.8	1870.3786	153.3097	.8	2273.2879	169.0177	.8	2715.4670	184.7256
.9	1878.0519	153.6239	.9	2281.7466	169.3318	.9	2724.7112	185.0398
49.0	1885.7409	153.9380	54.0	2290.2210	169.6460	59.0	2733.9710	185.3540
.1	1893.4457	154.2522	.1	2298.7112	169.9602	.1	2743.2466	185.6681
.2	1901.1662	154.5664	.2	2307.2171	170.2743	.2	2752.5378	185.9823
.3	1908.9024	154.8805	.3	2315.7386	170.5885	.3	2761.8448	186.2964
.4	1916.6543	155.1947	.4	2324.2759	170.9026	.4	2771.1675	186.6106
.5	1924.4218	155.5088	.5	2332.8289	171.2168	.5	2780.5058	186.9248
.6	1932.2051	155.8230	.6	2341.3976	171.5310	.6	2789.8599	187.2389
.7	1940.0042	156.1372	.7	2349.9820	171.8451	.7	2799.2297	187.5531
.8	1947.8189	156.4513	.8	2358.5821	172.1593	.8	2808.6152	187.8672
.9	1955.6493	156.7655	.9	2367.1979	172.4735	.9	2818.0165	188.1814

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
60.0	2827.4334	188.4956	65.0	3318.3072	204.2035	70.0	3848.4510	219.9115
.1	2836.8660	188.8097	.1	3328.5253	204.5176	.1	3859.4544	220.2256
.2	2846.3144	189.1239	.2	3338.7590	204.8318	.2	3870.4736	220.5398
.3	2855.7784	189.4380	.3	3349.0085	205.1460	.3	3881.5084	220.8540
.4	2865.2582	189.7522	.4	3359.2736	205.4602	.4	3892.5590	221.1681
.5	2874.7536	190.0664	.5	3369.5545	205.7743	.5	3903.6252	221.4823
.6	2884.2648	190.3805	.6	3379.8510	206.0885	.6	3914.7072	221.7964
.7	2893.7917	190.6947	.7	3390.1633	206.4026	.7	3925.8049	222.1106
.8	2903.3343	191.0088	.8	3400.4913	206.7168	.8	3936.9182	222.4248
.9	2912.8926	191.3230	.9	3410.8350	207.0310	.9	3948.0473	222.7389
61.0	2922.4666	191.6372	66.0	3421.1944	207.3451	71.0	3959.1921	223.0531
.1	2932.0563	191.9513	.1	3431.5695	207.6593	.1	3970.3526	223.3672
.2	2941.6617	192.2655	.2	3441.9603	207.9734	.2	3981.5289	223.6814
.3	2951.2828	192.5796	.3	3452.3669	208.2876	.3	3992.7208	223.9956
.4	2960.9197	192.8938	.4	3462.7891	208.6017	.4	4003.9284	224.3097
.5	2970.5722	193.2079	.5	3473.2270	208.9159	.5	4015.1518	224.6239
.6	2980.2405	193.5221	.6	3483.6807	209.2301	.6	4026.3908	224.9380
.7	2989.9244	193.8363	.7	3494.1500	209.5442	.7	4037.6456	225.2522
.8	2999.6241	194.1504	.8	3504.6351	209.8584	.8	4048.9160	225.5664
.9	3009.3395	194.4646	.9	3515.1359	210.1725	.9	4060.2022	225.3805
62.0	3019.0705	194.7787	67.0	3525.6524	210.4867	72.0	4071.5041	226.1947
.1	3028.8173	195.0929	.1	3536.1845	210.8009	.1	4082.8217	226.5088
.2	3038.5798	195.4071	.2	3546.7324	211.1150	.2	4094.1550	226.8230
.3	3048.3580	195.7212	.3	3557.2960	211.4292	.3	4105.5040	227.1371
.4	3058.1520	196.0354	.4	3567.8754	211.7433	.4	4116.8687	227.4513
.5	3067.9616	196.3495	.5	3578.4704	212.0575	.5	4128.2491	227.7655
.6	3077.7869	196.6637	.6	3589.0811	212.3717	.6	4139.6452	228.0796
.7	3087.6279	196.9779	.7	3599.7075	212.6858	.7	4151.0571	228.3938
.8	3097.4847	197.2920	.8	3610.3497	213.0000	.8	4162.4846	228.7079
.9	3107.3571	197.6062	.9	3621.0075	213.3141	.9	4173.9279	229.0221
63.0	3117.2453	197.9203	68.0	3631.6811	213.6283	73.0	4185.3868	229.3363
.1	3127.1492	198.2345	.1	3642.3704	213.9425	.1	4196.8615	229.6504
.2	3137.0688	198.5487	.2	3653.0754	214.2566	.2	4208.3519	229.9646
.3	3147.0040	198.8628	.3	3663.7960	214.5708	.3	4219.8579	230.2787
.4	3156.9550	199.1770	.4	3674.5324	214.8849	.4	4231.3797	230.5929
.5	3166.9217	199.4911	.5	3685.2845	215.1991	.5	4242.9172	230.9071
.6	3176.9043	199.8053	.6	3696.0523	215.5133	.6	4254.4704	231.2212
.7	3186.9023	200.1195	.7	3706.8359	215.8274	.7	4266.0394	231.5354
.8	3196.9161	200.4336	.8	3717.6351	216.1416	.8	4277.6240	231.8495
.9	3206.9456	200.7478	.9	3728.4500	216.4556	.9	4289.2243	232.1637
64.0	3216.9909	201.0620	69.0	3739.2807	216.7699	74.0	4300.8403	232.4779
.1	3227.0518	201.3761	.1	3750.1270	217.0841	.1	4312.4721	232.7920
.2	3237.1285	201.6902	.2	3760.9891	217.3982	.2	4324.1195	233.1062
.3	3247.2222	202.0044	.3	3771.8668	217.7124	.3	4335.7827	233.4203
.4	3257.3289	202.3186	.4	3782.7603	218.0265	.4	4347.4616	233.7345
.5	3267.4527	202.6327	.5	3793.6695	218.3407	.5	4359.1562	234.0487
.6	3277.5922	202.9469	.6	3804.5944	218.6548	.6	4370.8664	234.3628
.7	3287.7474	203.2610	.7	3815.5350	218.9690	.7	4382.5924	234.6770
.8	3297.9183	203.5752	.8	3826.4913	219.2832	.8	4394.3341	234.9911
.9	3308.1049	203.8894	.9	3837.4633	219.5973	.9	4406.0916	235.3053

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
75.0	4417.8647	235.6194	80.0	5026.5482	251.3274	85.0	5674.5017	267.0354
.1	4429.6535	235.9336	.1	5039.1225	251.6416	.1	5687.8614	267.3495
.2	4441.4580	236.2478	.2	5051.7124	251.9557	.2	5701.2367	267.6637
.3	4453.2783	236.5619	.3	5064.3180	252.2699	.3	5714.6277	267.9779
.4	4465.1142	236.8761	.4	5076.9394	252.5840	.4	5728.0345	268.2920
.5	4476.9659	237.1902	.5	5089.5764	252.8982	.5	5741.4569	268.6062
.6	4488.8332	237.5044	.6	5102.2292	253.2124	.6	5754.8951	268.9203
.7	4500.7163	237.8186	.7	5114.8977	253.5265	.7	5768.3490	269.2345
.8	4512.6151	238.1327	.8	5127.5819	253.8407	.8	5781.8185	269.5486
.9	4524.5296	238.4469	.9	5140.2818	254.1548	.9	5795.3038	269.8628
76.0	4536.4598	238.7610	81.0	5152.9973	254.4690	86.0	5808.8048	270.1770
.1	4548.4057	239.0752	.1	5165.7287	254.7832	.1	5822.3215	270.4911
.2	4560.3673	239.3894	.2	5178.4757	255.0973	.2	5835.8539	270.8053
.3	4572.3446	239.7035	.3	5191.2384	255.4115	.3	5849.4020	271.1194
.4	4584.3377	240.0177	.4	5204.0168	255.7256	.4	5862.9659	271.4336
.5	4596.3464	240.3318	.5	5216.8110	256.0398	.5	5876.5454	271.7478
.6	4608.3708	240.6460	.6	5229.6208	256.3540	.6	5890.1407	272.0619
.7	4620.4110	240.9602	.7	5242.4463	256.6681	.7	5903.7516	272.3761
.8	4632.4669	241.2743	.8	5255.2876	256.9823	.8	5917.3783	272.6902
.9	4644.5384	241.5885	.9	5268.1446	257.2966	.9	5931.0206	273.0044
77.0	4656.6257	241.9026	82.0	5281.0173	257.6106	87.0	5944.6787	273.3186
.1	4668.7287	242.2168	.1	5293.9056	257.9247	.1	5958.3525	273.6327
.2	4680.8474	242.5310	.2	5306.8097	258.2389	.2	5972.0420	273.9469
.3	4692.9818	242.8451	.3	5319.7295	258.5531	.3	5985.7472	274.2610
.4	4705.1319	243.1592	.4	5332.6650	258.8672	.4	5999.4681	274.5752
.5	4717.2977	243.4734	.5	5345.6162	259.1814	.5	6013.2047	274.8894
.6	4729.4792	243.7876	.6	5358.5832	259.4956	.6	6026.9570	275.2035
.7	4741.6765	244.1017	.7	5371.5658	259.8097	.7	6040.7250	275.5177
.8	4753.8894	244.4159	.8	5384.5641	260.1239	.8	6054.5088	275.8318
.9	4766.1181	244.7301	.9	5397.5782	260.4380	.9	6068.3082	276.1460
78.0	4778.3624	245.0442	83.0	5410.6079	260.7522	88.0	6082.1234	276.4602
.1	4790.6225	245.3584	.1	5423.6534	261.0663	.1	6095.9542	276.7743
.2	4802.8983	245.6725	.2	5436.7146	261.3805	.2	6109.8008	277.0885
.3	4815.1897	245.9867	.3	5449.7915	261.6947	.3	6123.6631	277.4026
.4	4827.4969	246.3009	.4	5462.8840	262.0088	.4	6137.5411	277.7168
.5	4839.8198	246.6150	.5	5475.9923	262.3230	.5	6151.4348	278.0309
.6	4852.1584	246.9292	.6	5489.1163	262.6371	.6	6165.3442	278.3451
.7	4864.5128	247.2433	.7	5502.2561	262.9513	.7	6179.2693	278.6593
.8	4876.8828	247.5575	.8	5515.4115	263.2655	.8	6193.2101	278.9740
.9	4889.2685	247.8717	.9	5528.5826	263.5796	.9	6207.1666	279.2876
79.0	4901.6699	248.1858	84.0	5541.7694	263.8938	89.0	6221.1389	279.6017
.1	4914.0871	248.5000	.1	5554.9720	264.2079	.1	6235.1268	279.9159
.2	4926.5199	248.8141	.2	5568.1902	264.5221	.2	6249.1304	280.2301
.3	4938.9685	249.1283	.3	5581.4242	264.8363	.3	6263.1498	280.5442
.4	4951.4328	249.4425	.4	5594.6739	265.1514	.4	6277.1849	280.8584
.5	4963.9127	249.7566	.5	5607.9392	265.4646	.5	6291.2356	281.1725
.6	4976.4084	250.0708	.6	5621.2203	265.7787	.6	6305.3021	281.4867
.7	4988.9198	250.3850	.7	5634.5171	266.0929	.7	6319.3843	281.8009
.8	5001.4469	250.6991	.8	5647.8296	266.4071	.8	6333.4822	282.1150
.9	5013.9897	251.0133	.9	5661.1578	266.7212	.9	6347.5958	282.4292

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
90.0	6361.7251	282.7433	93.5	6866.1471	293.7389	97.0	7389.8113	304.7345
.1	6375.8701	283.0575	.6	6880.8419	294.0531	.1	7405.0559	305.0486
.2	6390.0309	283.3717	.7	6895.5524	294.3672	.2	7420.3162	305.3628
.3	6404.2073	283.6858	.8	6910.2786	294.6814	.3	7435.5922	305.6770
.4	6418.3995	284.0000	.9	6925.0205	294.9956	.4	7450.8839	305.9911
.5	6432.6073	284.3141	94.0	6939.7782	295.3097	.5	7466.1913	306.3053
.6	6446.8309	284.6283	.1	6954.5515	295.6239	.6	7481.5144	306.6194
.7	6461.0701	284.9425	.2	6969.3106	295.9380	.7	7496.8532	306.9336
.8	6475.3251	285.2566	.3	6984.1453	296.2522	.8	7512.2078	307.2478
.9	6489.5958	285.5708	.4	6998.9658	296.5663	.9	7527.5780	307.5619
91.0	6503.8822	285.8849	.5	7013.8019	296.8805	98.0	7542.9640	307.8761
.1	6518.1843	286.1991	.6	7028.6538	297.1947	.1	7558.3656	308.1902
.2	6532.5021	286.5133	.7	7043.5214	297.5088	.2	7573.7830	308.5044
.3	6546.8356	286.8274	.8	7058.4047	297.8230	.3	7589.2161	308.8186
.4	6561.1848	287.1416	.9	7073.3033	298.1371	.4	7604.6648	309.1327
.5	6575.5498	287.4557	95.0	7088.2184	298.4513	.5	7620.1293	309.4469
.6	6589.9304	287.7699	.1	7103.1488	298.7655	.6	7635.6095	309.7610
.7	6604.3268	288.0840	.2	7118.1950	299.0796	.7	7651.1054	310.0752
.8	6618.7388	288.3982	.3	7133.0568	299.3938	.8	7666.6170	310.3894
.9	6633.1666	288.7124	.4	7148.0343	299.7079	.9	7682.1444	310.7035
92.0	6647.6101	289.0265	.5	7163.0276	300.0221	99.0	7697.6893	311.0177
.1	6662.0692	289.3407	.6	7178.0366	300.3363	.1	7713.2461	311.3318
.2	6676.5441	289.6548	.7	7193.0612	300.6504	.2	7728.8206	311.6460
.3	6691.0347	289.9690	.8	7208.1016	300.9646	.3	7744.4107	311.9602
.4	6705.5410	290.2832	.9	7223.1577	301.2787	.4	7760.0166	312.2743
.5	6720.0630	290.5973	96.0	7238.2295	301.5929	.5	7775.6382	312.5885
.6	6734.6008	290.9115	.1	7253.3170	301.9071	.6	7791.2754	312.9026
.7	6749.1542	291.2256	.2	7268.4202	302.2212	.7	7806.9284	313.2168
.8	6763.7233	291.5398	.3	7283.5391	302.5354	.8	7822.5971	313.5309
.9	6778.3082	291.8540	.4	7298.6737	302.8405	.9	7838.2815	313.8451
93.0	6792.9087	292.1681	.5	7313.8240	303.1637	100.0	7853.9816	314.1593
.1	6807.5250	292.4823	.6	7328.9901	303.4779			
.2	6822.1569	292.7964	.7	7344.1718	303.7920			
.3	6836.8046	293.1106	.8	7359.3693	304.1062			
.4	6851.4680	293.4248	.9	7374.5824	304.4203			

AREAS OF CIRCLES.

(Advancing by Eighths.)

AREAS.

Dia.	0.0	$0.\frac{1}{8}$	$0.\frac{1}{4}$	$0.\frac{3}{8}$	$0.\frac{1}{2}$	$0.\frac{5}{8}$	$0.\frac{3}{4}$	$0.\frac{7}{8}$
0	0.0	0.0122	0.0490	0.1104	0.1963	0.3068	0.4417	0.6013
1	0.7854	0.9940	1.227	1.484	1.767	2.073	2.405	2.761
2	3.1416	3.546	3.976	4.430	4.908	5.411	5.939	6.491
3	7.068	7.669	8.295	8.946	9.621	10.32	11.04	11.79
4	12.56	13.36	14.18	15.03	15.90	16.80	17.72	18.66
5	19.63	20.62	21.64	22.69	23.75	24.85	25.96	27.10
6	28.27	29.46	30.67	31.91	33.18	34.47	35.78	37.12
7	38.48	39.87	41.28	42.71	44.17	45.66	47.17	48.70
8	50.26	51.84	53.45	55.08	56.74	58.42	60.13	61.86
9	63.61	65.39	67.20	69.02	70.88	72.75	74.66	76.58
10	78.54	80.51	82.51	84.54	86.59	88.66	90.76	92.88
11	95.03	97.20	99.40	101.6	103.8	106.1	108.4	110.7
12	113.0	115.4	117.8	120.2	122.7	125.1	127.6	130.1
13	132.7	135.2	137.8	140.5	143.1	145.8	148.4	151.2
14	153.9	156.6	159.4	162.2	165.1	167.9	170.8	173.7
15	176.7	179.6	182.6	185.6	188.6	191.7	194.8	197.9
16	201.0	204.2	207.3	210.5	213.8	217.0	220.3	223.6
17	226.9	230.3	233.7	237.1	240.5	243.9	247.4	250.9
18	254.4	258.0	261.5	265.1	268.8	272.4	276.1	279.8
19	283.5	287.2	291.0	294.8	298.6	302.4	306.3	310.2
20	314.1	318.1	322.0	326.0	330.0	334.1	338.1	342.2
21	346.3	350.4	354.6	358.8	363.0	367.2	371.5	375.8
22	380.1	384.4	388.8	393.2	397.6	402.0	406.4	410.9
23	415.4	420.0	424.5	429.1	433.7	438.3	443.0	447.6
24	452.3	457.1	461.8	466.6	471.4	476.2	481.1	485.9
25	490.8	495.7	500.7	505.7	510.7	515.7	520.7	525.8
26	530.9	536.0	541.1	546.3	551.5	556.7	562.0	567.2
27	572.5	577.8	583.2	588.5	593.9	599.3	604.8	610.2
28	615.7	621.2	626.7	632.3	637.9	643.5	649.1	654.8
29	660.5	666.2	671.9	677.7	683.4	689.2	695.1	700.9
30	706.8	712.7	718.6	724.6	730.6	736.6	742.6	748.6
31	754.8	760.9	767.0	773.1	779.3	785.5	791.7	798.0
32	804.3	810.6	816.9	823.2	829.6	836.0	842.4	848.8
33	855.3	861.8	868.3	874.9	881.4	888.0	894.6	901.3
34	907.9	914.7	921.3	928.1	934.8	941.6	948.4	955.3
35	962.1	969.0	975.9	982.8	989.8	996.8	1003.8	1010.8
36	1017.9	1025.0	1032.1	1039.2	1046.3	1053.5	1060.7	1068.0
37	1075.2	1082.5	1089.8	1097.1	1104.5	1111.8	1119.2	1126.7
38	1134.1	1141.6	1149.1	1156.6	1164.2	1171.7	1179.3	1186.9
39	1194.6	1202.3	1210.0	1217.7	1225.4	1233.2	1241.0	1248.8
40	1256.6	1264.5	1272.4	1280.3	1288.2	1296.2	1304.2	1312.2
41	1320.3	1328.3	1336.4	1344.5	1352.7	1360.8	1369.0	1377.2
42	1385.4	1393.7	1402.0	1410.3	1418.6	1427.0	1435.4	1443.8
43	1452.2	1460.7	1469.1	1477.6	1486.2	1494.7	1503.3	1511.9
44	1520.5	1529.2	1537.9	1546.6	1555.3	1564.0	1572.8	1581.6
45	1590.4	1599.3	1608.2	1617.0	1626.0	1634.9	1643.9	1652.9

CIRCUMFERENCES OF CIRCLES.

(Advancing by Eighths.)

CIRCUMFERENCES.

Dia.	0.0	$0.\frac{1}{8}$	$0.\frac{1}{4}$	$0.\frac{3}{8}$	$0.\frac{1}{2}$	$0.\frac{5}{8}$	$0.\frac{3}{4}$	$0.\frac{7}{8}$
0	0.0	0.3927	0.7854	1.178	1.570	1.963	2.356	2.748
1	3.141	3.534	3.927	4.319	4.712	5.105	5.497	5.890
2	6.283	6.675	7.068	7.461	7.854	8.246	8.639	9.032
3	9.424	9.817	10.21	10.60	10.99	11.38	11.78	12.17
4	12.56	12.95	13.35	13.74	14.13	14.52	14.92	15.31
5	15.70	16.10	16.49	16.88	17.27	17.67	18.06	18.45
6	18.84	19.24	19.63	20.02	20.42	20.81	21.20	21.59
7	21.99	22.38	22.77	23.16	23.56	23.95	24.34	24.74
8	25.13	25.52	25.91	26.31	26.70	27.09	27.48	27.88
9	28.27	28.66	29.05	29.45	29.84	30.23	30.63	31.02
10	31.41	31.80	32.20	32.59	32.98	33.37	33.77	34.16
11	34.55	34.95	35.34	35.73	36.12	36.52	36.91	37.30
12	37.69	38.09	38.48	38.87	39.27	39.66	40.05	40.44
13	40.84	41.23	41.62	42.01	42.41	42.80	43.19	43.58
14	43.98	44.37	44.76	45.16	45.55	45.94	46.33	46.73
15	47.12	47.51	47.90	48.30	48.69	49.08	49.48	49.87
16	50.26	50.65	51.05	51.44	51.83	52.22	52.62	53.01
17	53.40	53.79	54.19	54.58	54.97	55.37	55.76	56.15
18	56.54	56.94	57.33	57.72	58.11	58.51	58.90	59.29
19	59.69	60.08	60.47	60.86	61.26	61.65	62.04	62.43
20	62.83	63.22	63.61	64.01	64.40	64.79	65.18	65.58
21	65.97	66.36	66.75	67.15	67.54	67.93	68.32	68.72
22	69.11	69.50	69.90	70.29	70.68	71.07	71.47	71.86
23	72.25	72.64	73.04	73.43	73.82	74.22	74.61	75.00
24	75.39	75.79	76.18	76.57	76.96	77.36	77.75	78.14
25	78.54	78.93	79.32	79.71	80.10	80.50	80.89	81.28
26	81.68	82.07	82.46	82.85	83.25	83.64	84.03	84.43
27	84.82	85.21	85.60	86.00	86.39	86.78	87.17	87.57
28	87.96	88.35	88.75	89.14	89.53	89.92	90.32	90.71
29	91.10	91.49	91.89	92.28	92.67	93.06	93.46	93.85
30	94.24	94.64	95.03	95.42	95.81	96.21	96.60	96.99
31	97.39	97.78	98.17	98.57	98.96	99.35	99.75	100.14
32	100.53	100.92	101.32	101.71	102.10	102.49	102.89	103.29
33	103.67	104.07	104.46	104.85	105.24	105.64	106.03	106.42
34	106.81	107.21	107.60	107.99	108.39	108.78	109.17	109.56
35	109.96	110.35	110.74	111.13	111.53	111.92	112.31	112.71
36	113.10	113.49	113.88	114.28	114.67	115.06	115.45	115.85
37	116.24	116.63	117.02	117.42	117.81	118.20	118.60	118.99
38	119.38	119.77	120.17	120.56	120.95	121.34	121.74	122.13
39	122.52	122.92	123.31	123.70	124.09	124.49	124.88	125.27
40	125.66	126.06	126.45	126.84	127.24	127.63	128.02	128.41
41	128.81	129.20	127.59	129.98	130.38	130.77	131.16	131.55
42	131.95	132.34	132.73	133.13	133.52	133.91	134.30	134.70
43	135.09	135.48	135.87	136.27	136.66	137.05	137.45	137.84
44	138.23	138.62	139.02	139.41	139.80	140.19	140.59	140.98
45	141.37	141.76	142.16	142.55	142.94	143.34	143.73	144.12

AREAS AND CIRCUMFERENCES OF CIRCLES.

FROM 1 TO 50 FEET.

(Advancing by One Inch.)

Dia.	Area.	Circum.		Dia.	Area.	Circum.		Dia.	Area.	Circum.	
Ft.	Feet.	Ft.	In.	Feet	Feet.	Ft.	In.	Ft.	Feet.	Ft.	In.
1 0	0.7854	3	1 5/8	5 0	19.635	15	8 1/8	9 0	63.6174	28	3 1/4
1	0.9217	3	4 5/8	1	20.2947	15	11 5/8	1	64.8006	28	6 3/8
2	1.069	3	8	2	20.9656	16	2 3/4	2	65.9951	28	9 1/2
3	1.2271	3	11	3	21.6475	16	5 3/4	3	67.2007	29	5/8
4	1.3962	4	2 1/8	4	22.34	16	9	4	68.4166	29	3 3/4
5	1.5761	4	5 3/8	5	23.0437	17	1/8	5	69.644	29	7
6	1.7671	4	8 1/2	6	23.7583	17	3 1/4	6	70.8823	29.	10 1/8
7	1.9689	4	11 5/8	7	24.4835	17	6 3/8	7	72.1309	30	1 1/4
8	2.1816	5	2 3/4	8	25.2199	17	9 5/8	8	73.391	30	4 3/8
9	2.4052	5	5 7/8	9	25.9672	18	3/4	9	74.662	30	7 1/2
10	2.6398	5	9	10	26.7251	18	3 7/8	10	75.9433	30	11 5/8
11	2.8852	6	1/4	11	27.4943	18	7 1/8	11	77.2362	31	1 3/4
2 0	3.1416	6	3 3/8	6 0	28.2744	18	10 1/8	10 0	78.54	31	5
1	3.4087	6	6 1/2	1	29.0649	19	1 1/4	1	79.854	31	8 1/8
2	3.6869	6	9 5/8	2	29.8668	19	4 3/8	2	81.1795	31	11 1/4
3	3.976	7	3/4	3	30.6796	19	7 1/2	3	82.516	32	2 3/8
4	4.276	7	3 7/8	4	31.5029	19	10 5/8	4	83.8627	32	5 1/2
5	4.5869	7	7	5	32.3376	20	1 7/8	5	85.2211	32	8 5/8
6	4.9087	7	10 1/4	6	33.1831	20	4 7/8	6	86.5903	32	11 3/4
7	5.2413	8	1 3/8	7	34.0391	20	8 1/8	7	87.9697	33	2 7/8
8	5.585	8	4 1/2	8	34.9065	20	11 1/2	8	89.3608	33	6 1/8
9	5.9395	8	7 5/8	9	35.7847	21	2 3/8	9	90.7627	33	9 1/4
10	6.3049	8	10 3/4	10	36.6735	21	5 1/8	10	92.1749	34	3/8
11	6.6813	9	1 7/8	11	37.5736	21	8 3/4	11	93.5986	34	3 1/2
3 0	7.0686	9	5	7 0	38.4846	21	11 7/8	11 0	95.0334	34	6 5/8
1	7.4666	9	8 1/4	1	39.406	22	3	1	96.4783	34	9 3/4
2	7.8757	9	11 3/8	2	40.3388	22	6 1/8	2	97.9347	35	7/8
3	8.2957	10	2 1/2	3	41.2825	22	9 1/4	3	99.4021	35	4 1/8
4	8.7265	10	5 5/8	4	42.2367	23	3/8	4	100.8797	35	7 1/4
5	9.1683	10	8 3/4	5	43.2022	23	2 1/8	5	102.3689	35	10 5/8
6	9.6211	10	11 7/8	6	44.1787	23	6 3/4	6	103.8691	36	1 1/2
7	10.0846	11	3	7	45.1656	23	9 7/8	7	105.3794	36	4 1/2
8	10.5591	11	6 1/8	8	46.1638	24	1 1/8	8	106.9013	36	7 3/4
9	11.0446	11	9 3/8	9	47.173	24	4 1/8	9	108.4342	36	10 7/8
10	11.5409	12	1/2	10	48.1962	24	7 1/4	10	109.9772	37	2 3/4
11	12.0481	12	3 5/8	11	49.2236	24	10 3/8	11	111.5319	37	5 1/4
4 0	12.5664	12	6 3/4	8 0	50.2656	25	1 1/2	12 0	113.0976	37	8 3/4
1	13.0952	12	9 7/8	1	51.3178	25	4 5/8	1	114.6732	37	11 1/2
2	13.6353	13	1	2	52.3816	25	7 7/8	2	116.2607	38	2 5/8
3	14.1862	13	4 1/8	3	53.4562	25	11	3	117.859	38	5 3/4
4	14.7479	13	7 1/4	4	54.5412	26	2 1/8	4	119.4674	38	8 7/8
5	15.3206	13	10 1/2	5	55.6377	26	5 1/4	5	121.0876	39	0
6	15.9043	14	1 5/8	6	56.7451	26	8 3/8	6	122.7187	39	3 1/4
7	16.4986	14	4 5/8	7	57.8628	26	11 1/2	7	124.3598	39	6 3/8
8	17.1041	14	7 7/8	8	58.992	27	2 3/4	8	126.0127	39	9 1/2
9	17.7205	14	11	9	60.1321	27	5 3/4	9	127.6765	40	5/8
10	18.3476	15	2 1/8	10	61.2826	27	9	10	129.3504	40	3 3/4
11	18.9858	15	5 1/4	11	62.4445	28	1/8	11	131.036	40	6 7/8

Areas and Circumferences of Circles (Feet and Inches).

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
Ft.	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*
13 0	132.7326	40 10	18 0	254.4696	56 6½	23 0	415.4766	72 3
1	134.4391	41 1⅛	1	256.8303	56 9⅝	1	418.4915	72 6⅛
2	136.1574	41 4⅜	2	259.2033	57 ⅞	2	421.5192	72 9⅜
3	137.8867	41 7½	3	261.5872	57 4	3	424.5577	73 ½
4	139.626	41 10⅝	4	263.9807	57 7⅛	4	427.6055	73 3⅝
5	141.3771	42 1⅝	5	266.3864	57 10¼	5	430.6658	73 6¾
6	143.1391	42 4⅞	6	268.8031	58 1⅜	6	433.7371	73 9⅞
7	144.9111	42 8	7	271.2293	58 4½	7	436.8175	74 1
8	146.6949	42 11⅛	8	273.6678	58 7⅝	8	439.9106	74 4⅛
9	148.4896	43 2¼	9	276.1171	58 10¾	9	443.0146	74 7¼
10	150.2943	43 5½	10	278.5761	58 2	10	446.1278	74 10⅝
11	152.1109	43 8⅝	11	281.0472	59 5⅛	11	449.2536	75 1⅝
14 0	153.9384	43 11¾	19 0	283.5294	59 8¼	24 0	452.3904	75 4¾
1	155.7758	44 2⅞	1	286.021	59 11½	1	455.5362	75 7⅞
2	157.625	44 6	2	288.5249	60 2½	2	458.6948	75 11
3	159.4852	44 9⅛	3	291.0397	60 5⅝	3	461.8642	76 2⅛
4	161.3553	45 ¼	4	293.5641	60 8¾	4	465.0428	76 5¼
5	163.2373	45 3½	5	296.1107	60 11⅞	5	468.2341	76 8½
6	165.1303	45 6⅝	6	298.6483	60 3⅛	6	471.4363	76 11⅝
7	167.0331	45 9¾	7	301.2054	61 6¼	7	474.6476	77 2¾
8	168.9479	46 ⅞	8	303.7747	61 9½	8	477.8716	77 5⅞
9	170.8735	46 4	9	306.355	61 ½	9	481.1065	77 9
10	172.8091	46 7⅛	10	308.9448	61 3⅝	10	484.3506	78 ⅛
11	174.7565	46 11¼	11	311.5469	62 6¾	11	487.6073	78 3¼
15 0	176.715	47 1½	20 0	314.16	62 9⅞	25 0	490.875	78 6½
1	178.6832	47 4⅝	1	316.7824	62 1⅛	1	494.1516	78 9½
2	180.6634	47 7¾	2	319.4173	63 4¼	2	497.4411	79 ¾
3	182.6545	47 10⅞	3	322.063	63 7⅜	3	500.7415	79 3⅞
4	184.6555	48 2½	4	324.7182	63 11½	4	504.051	79 7⅛
5	186.6684	48 5⅛	5	327.3858	63 1⅝	5	507.3732	79 11⅛
6	188.6923	48 8¼	6	330.0643	64 4¾	6	510.7063	80 1⅜
7	190.726	48 11⅜	7	332.7522	64 7⅞	7	514.0484	80 4½
8	192.7716	49 2⅝	8	335.4525	64 11	8	517.4034	80 7⅝
9	194.8282	49 5¾	9	338.1637	65 2¼	9	520.7692	80 10¾
10	196.8946	49 8⅞	10	340.8844	65 5⅜	10	524.1441	81 1⅞
11	198.973	50 0	11	343.6174	65 8¼	11	527.5318	81 5
16 0	201.0624	50 3⅛	21 0	346.3614	65 11⅝	26 0	530.9304	81 8⅛
1	203.1615	50 6¼	1	349.1147	66 2¾	1	534.3379	81 11¼
2	205.2726	50 9⅝	2	351.8804	66 5⅞	2	537.7583	82 2⅜
3	207.3946	51 ½	3	354.6571	66 9	3	541.1896	82 5¼
4	209.5264	51 3¾	4	357.4432	66 ⅛	4	544.6299	82 8¾
5	211.6703	51 6½	5	360.2417	67 3⅜	5	548.083	82 11⅞
6	213.8251	51 10	6	363.0511	67 6½	6	551.5471	83 3
7	215.9896	52 1⅛	7	365.8698	67 9⅝	7	555.0201	83 6⅛
8	218.1662	52 4¼	8	368.7011	68 ¾	8	558.5059	83 9¼
9	220.3537	52 7⅜	9	371.5432	68 3⅞	9	562.0027	84 ⅜
10	222.551	52 10½	10	374.3947	68 7	10	565.5084	84 3½
11	224.7608	53 1⅝	11	377.2587	68 10¼	11	569.027	84 6⅝
17 0	226.9806	53 4⅞	22 0	380.1336	69 1⅜	27 0	572.5566	84 9⅞
1	229.2105	53 8	1	383.0177	69 4½	1	576.0949	85 1
2	231.4525	53 11⅛	2	385.9144	69 7⅝	2	579.6463	85 4¼
3	233.7055	54 2⅛	3	388.822	69 10¾	3	583.2085	85 8¼
4	235.9682	54 5⅜	4	391.7389	70 1⅞	4	586.7796	85 11⅜
5	238.243	54 8½	5	394.6683	70 5	5	590.3637	86 1½
6	240.5287	54 11⅝	6	397.6087	70 8¼	6	593.9587	86 4⅝
7	242.8241	55 2⅞	7	400.5583	70 11⅛	7	597.5625	86 7⅞
8	245.1316	55 6	8	403.5204	71 2½	8	601.1793	86 11
9	247.45	55 9⅛	9	406.4935	71 5⅝	9	604.807	87 2⅛
10	249.7781	56 ¼	10	409.4759	71 8¾	10	608.4436	87 5¼
11	252.1184	56 3½	11	412.4707	71 11⅞	11	612.0931	87 8¾

Areas and Circumferences of Circles (Feet and Inches).

Dia.	Area.	Circum.	Dia.	Area.	Circum.	Dia.	Area.	Circum.
Ft.	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*
28 0	615.7536	87 11½	33 0	855.301	103 8	38 0	1134.118	119 4½
1	619.4228	88 2⅝	1	859.624	103 11⅛	1	1139.095	119 7⅝
2	623.105	88 5¾	2	863.961	104 2¼	2	1144.087	119 10¾
3	626.7982	88 9	3	868.309	104 5⅜	3	1149.089	120 2
4	630.5002	89 ⅛	4	872.665	104 8⅝	4	1154.110	120 5⅛
5	634.2152	89 3¼	5	877.035	104 11¾	5	1159.124	120 8⅜
6	637.9411	89 6⅜	6	881.415	105 2⅞	6	1164.159	120 11⅜
7	641.6758	89 9½	7	885.804	105 6	7	1169.202	121 2½
8	645.4235	90 ⅝	8	890.206	105 9⅛	8	1174.259	121 5⅝
9	649.1821	90 3¾	9	394.619	106 ¼	9	1179.327	121 8¾
10	652.9495	90 6⅛	10	899.041	106 3⅜	10	1184.403	121 11⅞
11	656.73	90 11⅛	11	903.476	106 6⅝	11	1189.493	122 3⅛
29 0	660.5214	91 1¼	34 0	907.922	106 9¾	39 0	1194.593	122 6¼
1	664.3214	91 4⅜	1	912.377	107 ⅞	1	1199.719	122 9½
2	668.1346	91 7½	2	916.844	107 4	2	1204.824	123 ½
3	671.9587	91 10⅝	3	921.323	107 7⅛	3	1209.958	123 3⅝
4	675.7915	92 1¾	4	925.810	107 10½	4	1215.099	123 6¾
5	679.6375	92 4⅞	5	930.311	108 1⅜	5	1220.254	123 9⅞
6	683.4943	92 8⅛	6	934.822	108 4⅝	6	1225.420	124 1⅛
7	687.3598	92 11¼	7	939.342	108 7¾	7	1230.594	124 4¼
8	691.2385	93 2⅜	8	943.875	108 10⅞	8	1235.782	124 7⅜
9	695 1028	93 5⅛	9	948.419	109 2	9	1240.981	124 10½
10	699.0263	93 8⅝	10	952.972	109 5⅛	10	1246.188	125 1⅝
11	702.9377	93 11⅞	11	957.538	109 8¼	11	1251.408	125 4¾
30 0	706.86	94 2⅞	35 0	962.115	109 11⅜	40 0	1256.64	125 7⅞
1	710.791	94 6	1	966.770	110 2⅝	1	1261.879	125 11
2	714.735	94 9¼	2	971.299	110 5¾	2	1267.133	126 2¼
3	718.69	95 ⅜	3	975.908	110 8⅞	3	1272.397	126 5⅜
4	722.654	95 3½	4	980.526	111 0	4	1277.669	126 8½
5	726.631	95 6⅝	5	985.158	111 3⅛	5	1282.955	126 11⅝
6	730.618	95 9¾	6	989.803	111 6¼	6	1288.252	127 2¾
7	734.615	96 ⅞	7	994.451	111 9⅜	7	1293.557	127 5⅞
8	738.624	96 4	8	999.115	112 ½	8	1298.876	217 9
9	742.645	96 7¼	9	1003.79	112 3¾	9	1304.206	128 ½
10	746.674	96 10⅜	10	1008.473	112 6⅞	10	1309.543	128 3⅜
11	750.716	97 1½	11	1013.170	112 10	11	1314.895	128 6½
31 0	754.769	97 4⅝	36 0	1017.878	113 1⅛	41 0	1320.257	128 9⅝
1	758.831	97 7¾	1	1022.594	113 4¼	1	1325.628	129 ¾
2	762.906	97 10⅞	2	1027.324	113 7⅜	2	1331.012	129 3⅞
3	766.992	98 2	3	1032.064	113 10⅝	3	1336.407	129 7
4	771.086	98 5⅛	4	1036.813	114 1¾	4	1341.810	129 10⅛
5	775.191	98 8⅜	5	1041.576	114 4⅞	5	1347.227	130 1⅜
6	779.313	98 11½	6	1046.349	114 8	6	1352.655	130 4½
7	783.440	99 2⅝	7	1051.130	114 11⅛	7	1358.091	130 7⅝
8	787.581	99 5¾	8	1055.926	115 2¼	8	1363.541	130 10¾
9	791.732	99 8⅞	9	1060.731	115 5⅜	9	1369.001	131 1⅞
10	795.892	100 0	10	1065.546	115 9¼	10	1374.47	131 5
11	800.065	100 3⅛	11	1070.374	115 11⅝	11	1379.952	131 8⅛
32 0	804.25	100 6⅜	37 0	1075.2126	116 2⅞	42 0	1385.446	131 11⅜
1	808.442	100 9½	1	1080.059	116 6	1	1390.247	132 2½
2	812.648	101 ⅝	2	1084.920	116 9⅛	2	1396.462	132 5⅝
3	816.865	101 3¾	3	1089.791	117 ¼	3	1401.988	132 8¾
4	821.090	101 6⅞	4	1094.671	117 3½	4	1407.522	132 11⅞
5	825.329	101 10	5	1099.564	117 6½	5	1413.07	133 3
6	829.579	102 1⅛	6	1104.469	117 9⅝	6	1418.629	133 6⅛
7	833.837	102 4⅜	7	1109.381	118 ¾	7	1424.195	133 9¼
8	838.108	102 7½	8	1114.307	118 4	8	1429.776	134 ⅜
9	842.391	102 10⅝	9	1119.244	118 7⅛	9	1435.367	134 3⅝
10	846.681	103 1¾	10	1124.189	118 10¼	10	1440.967	134 6¾
11	850.985	103 4⅞	11	1129.148	119 1⅜	11	1446.580	134 9⅞

Areas and Circumferences of Circles (Feet and Inches).

Dia.	Area.	Circum.		Dia.	Area.	Circum.		Dia.	Area.	Circum.	
Ft.	*Feet.*	*Ft.*	*In.*	*Ft.*	*Feet.*	*Ft.*	*In.*	*Ft.*	*Feet.*	*Ft.*	*In.*
43 0	1452.205	135	1	46 0	1661.906	144	6⅛	49 0	1885.745	153	11¼
1	1457.836	135	4⅛	1	1667.931	144	9¼	1	1892.172	154	2⅜
2	1463.483	135	7¼	2	1673.97	145	⅜	2	1898.504	154	5½
3	1469.14	135	10½	3	1680.02	145	3½	3	1905.037	154	8⅝
4	1474.804	136	1⅝	4	1686.077	145	6⅝	4	1911.497	154	11⅞
5	1480.483	136	4¾	5	1692.148	145	9⅞	5	1917.961	155	2⅞
6	1486.173	136	7⅞	6	1698.231	146	1⅛	6	1924.426	155	6
7	1491.870	136	11	7	1704.321	146	4⅛	7	1930.919	155	9¼
8	1497.582	137	2⅛	8	1710.425	146	7¼	8	1937.316	156	⅛
9	1503.305	137	5¼	9	1716.541	146	10⅜	9	1943.914	156	3¼
10	1509.035	137	8⅜	10	1722.663	147	1½	10	1950.439	156	6⅝
11	1514.779	137	11⅝	11	1728.801	147	4⅝	11	1956.969	156	9¾
44 0	1520.534	138	2¾	47 0	1734.947	147	7¾	50 0	1963.5	157	⅞
1	1526.297	138	5⅞	1	1741.104	147	11				
2	1532.074	138	9	2	1747.274	148	2⅛				
3	1537.862	139	⅛	3	1753.455	148	5¼				
4	1543.658	139	3¼	4	1759.643	148	8⅜				
5	1549.478	139	6⅜	5	1765.845	148	11½				
6	1555.288	139	9⅝	6	1772.059	149	2⅝				
7	1561.116	140	¾	7	1778.28	149	5⅞				
8	1566.959	140	3⅞	8	1784.515	149	8⅞				
9	1572.812	140	7½	9	1790.761	150	⅛				
10	1578.673	141	10⅛	10	1797.015	150	3¼				
11	1584.549	141	1¼	11	1803.283	150	6⅜				
45 0	1590.435	141	4⅜	48 0	1809.562	150	9½				
1	1596.329	141	7½	1	1815.848	151	⅝				
2	1602.237	141	10⅔	2	1822.149	151	3¾				
3	1608.155	142	1⅞	3	1828.460	151	6⅞				
4	1614.082	142	5	4	1834.779	151	10⅛				
5	1620.023	142	8⅛	5	1841.173	152	1¼				
6	1625.974	142	11¼	6	1847.457	152	4⅜				
7	1631.933	143	2⅜	7	1853.809	152	7½				
8	1637.907	143	5½	8	1860.175	152	10⅝				
9	1643.891	143	8¾	9	1866.552	153	1¾				
10	1649.883	143	11⅞	10	1872.937	153	4⅞				
11	1555.889	144	3	11	1879.335	153	8⅛				

Circular Arcs.

To find the length of a circular arc when its chord and height, or versed sine is given; BY THE FOLLOWING TABLE.

RULE.—Divide the height by the chord; find in the column of heights the number equal to this quotient. Take out the corresponding number from the column of lengths. Multiply this number by the given chord.

EXAMPLE.—The chord of an arc is 80 and its versed sine is 30, what is the length of the arc?

Ans. $30 \div 80 = 0.375$. The length of an arc for a height of 0.375 we find from table to be 1.34063. $80 \times 1.34063 = 107.2504$ = length of arc.

TABLE OF CIRCULAR ARCS.

Hts.	Lengths	Hts.	Lengths	Hts.	Lengths	Hts.	Lengths	Hts.	Lengths
.001	1.00001	.062	1.01021	.123	1.03987	.184	1.08797	.245	1.15308
.002	1.00001	.063	1.01054	.124	1.04051	.185	1.08890	.246	1.15428
.003	1.00002	.064	1.01088	.125	1.04116	.186	1.08984	.247	1.15549
.004	1.00004	.065	1.01123	.126	1.04181	.187	1.09079	.248	1.15670
.005	1.00007	.066	1.01158	.127	1.04247	.188	1.09174	.249	1.15791
.006	1.00010	.067	1.01193	.128	1.04313	.189	1.09269	.250	1.15912
.007	1.00013	.068	1.01228	.129	1.04380	.190	1.09365	.251	1.16034
.008	1.00017	.069	1.01264	.130	1.04447	.191	1.09461	.252	1.16156
.009	1.00022	.070	1.01301	.131	1.04515	.192	1.09557	.253	1.16279
.010	1.00027	.071	1.01338	.132	1.04584	.193	1.09654	.254	1.16402
.011	1.00032	.072	1.01376	.133	1.04652	.194	1.09752	.255	1.16526
.012	1.00038	.073	1.01414	.134	1.04722	.195	1.09850	.256	1.16650
.013	1.00045	.074	1.01453	.135	1.04792	.196	1.09949	.257	1.16774
.014	1.00053	.075	1.01493	.136	1.04862	.197	1.10048	.258	1.16899
.015	1.00061	.076	1.01533	.137	1.04932	.198	1.10147	.259	1.17024
.016	1.00069	.077	1.01573	.138	1.05003	.199	1.10247	.260	1.17150
.017	1.00078	.078	1.01614	.139	1.05075	.200	1.10347	.261	1.17276
.018	1.00087	.079	1.01656	.140	1.05147	.201	1.10447	.262	1.17403
.019	1.00097	.080	1.01698	.141	1.05220	.202	1.10548	.263	1.17530
.020	1.00107	.081	1.01741	.142	1.05293	.203	1.10650	.264	1.17657
.021	1.00117	.082	1.01784	.143	1.05367	.204	1.10752	.265	1.17784
.022	1.00128	.083	1.01828	.144	1.05441	.205	1.10855	.266	1.17912
.023	1.00140	.084	1.01872	.145	1.05516	.206	1.10958	.267	1.18040
.024	1.00153	.085	1.01916	.146	1.05591	.207	1.11062	.268	1.18169
.025	1.00167	.086	1.01961	.147	1.05667	.208	1.11165	.269	1.18299
.026	1.00182	.087	1.02006	.148	1.05743	.209	1.11269	.270	1.18429
.027	1.00196	.088	1.02052	.149	1.05819	.210	1.11374	.271	1.18559
.028	1.00210	.089	1.02098	.150	1.05896	.211	1.11479	.272	1.18689
.029	1.00225	.090	1.02145	.151	1.05973	.212	1.11584	.273	1.18820
.030	1.00240	.091	1.02192	.152	1.06051	.213	1.11690	.274	1.18951
.031	1.00256	.092	1.02240	.153	1.06130	.214	1.11796	.275	1.19082
.032	1.00272	.093	1.02289	.154	1.06209	.215	1.11904	.276	1.19214
.033	1.00289	.094	1.02339	.155	1.06288	.216	1.12011	.277	1.19346
.034	1.00307	.095	1.02389	.156	1.06368	.217	1.12118	.278	1.19479
.035	1.00327	.096	1.02440	.157	1.06449	.218	1.12225	.279	1.19612
.036	1.00345	.097	1.02491	.158	1.06530	.219	1.12334	.280	1.19746
.037	1.00364	.098	1.02542	.159	1.06611	.220	1.12444	.281	1.19880
.038	1.00384	.099	1.02593	.160	1.06693	.221	1.12554	.282	1.20014
.039	1.00405	.100	1.02645	.161	1.06775	.222	1.12664	.283	1.20149
.040	1.00426	.101	1.02698	.162	1.06858	.223	1.12774	.284	1.20284
.041	1.00447	.102	1.02752	.163	1.06941	.224	1.12885	.285	1.20419
.042	1.00469	.103	1.02806	.164	1.07025	.225	1.12997	.286	1.20555
.043	1.00492	.104	1.02860	.165	1.07109	.226	1.13108	.287	1.20691
.044	1.00515	.105	1.02914	.166	1.07194	.227	1.13219	.288	1.20827
.045	1.00539	.106	1.02970	.167	1.07279	.228	1.13331	.289	1.20964
.046	1.00563	.107	1.03026	.168	1.07365	.229	1.13444	.290	1.21102
.047	1.00587	.108	1.03082	.169	1.07451	.230	1.13557	.291	1.21239
.048	1.00612	.109	1.03139	.170	1.07537	.231	1.13671	.292	1.21377
.049	1.00638	.110	1.03196	.171	1.07624	.232	1.13785	.293	1.21515
.050	1.00665	.111	1.03254	.172	1.07711	.233	1.13900	.294	1.21654
.051	1.00692	.112	1.03312	.173	1.07799	.234	1.14015	.295	1.21794
.052	1.00720	.113	1.03371	.174	1.07888	.235	1.14131	.296	1.21933
.053	1.00748	.114	1.03430	.175	1.07977	.236	1.14247	.297	1.22073
.054	1.00776	.115	1.03490	.176	1.08066	.237	1.14363	.298	1.22213
.055	1.00805	.116	1.03551	.177	1.08156	.238	1.14480	.299	1.22354
.056	1.00834	.117	1.03611	.178	1.08246	.239	1.14597	.300	1.22495
.057	1.00864	.118	1.03672	.179	1.08337	.240	1.14714	.301	1.22636
.058	1.00895	.119	1.03734	.180	1.08428	.241	1.14832	.302	1.22778
.059	1.00926	.120	1.03797	.181	1.08519	.242	1.14951	.303	1.22920
.060	1.00957	.121	1.03860	.182	1.08611	.243	1.15070	.304	1.23063
.061	1.00989	.122	1.03923	.183	1.08704	.244	1.15189	.305	1.23206

Table of Circular Arcs (*concluded*).

Hts.	Lengths	Hts.	Lengths	Hts.	Lengths	Hts.	Lengths	Hts.	Lengths
.306	1.23349	.345	1.29209	.384	1.35575	.423	1.42402	.462	1.49651
.307	1.23492	.346	1.29366	.385	1.35744	.424	1.42583	.463	1.49842
.308	1.23636	.347	1.29523	.386	1.35914	.425	1.42764	.464	1.50033
.309	1.23781	.348	1.29681	.387	1.36084	.426	1.42945	.465	1.50224
.310	1.23926	.349	1.29839	.388	1.36254	.427	1.43127	.466	1.50416
.311	1.24070	.350	1.29997	.389	1.36425	.428	1.43309	.467	1.50608
.312	1.24216	.351	1.30156	.390	1.36596	.429	1.43491	.468	1.50800
.313	1.24361	.352	1.30315	.391	1.36767	.430	1.43673	.469	1.50992
.314	1.24507	.353	1.30474	.392	1.36939	.431	1.43856	.470	1.51185
.315	1.24654	.354	1.30634	.393	1.37111	.432	1.44039	.471	1.51378
.316	1.24801	.355	1.30794	.394	1.37283	.433	1.44222	.472	1.51571
.317	1.24948	.356	1.30954	.395	1.37455	.434	1.44405	.473	1.51764
.318	1.25095	.357	1.31115	.396	1.37623	.435	1.44589	.474	1.51958
.319	1.25243	.358	1.31276	.397	1.37801	.436	1.44773	.475	1.52152
.320	1.25391	.359	1.31437	.398	1.37974	.487	1.44957	.476	1.52346
.321	1.25540	.360	1.31599	.399	1.38148	.438	1.45142	.477	1.52541
.322	1.25689	.361	1.31761	.400	1.38322	.439	1.45327	.478	1.52736
.323	1.25838	.362	1.31923	.401	1.38496	.440	1.45512	.479	1.52931
.324	1.25988	.363	1.32086	.402	1.38671	.441	1.45697	.480	1.53126
.325	1.26138	.364	1.32249	.403	1.38846	.442	1.45883	.481	1.53322
.326	1.26288	.365	1.32413	.404	1.39021	.443	1.46069	.482	1.53518
.327	1.26437	.366	1.32577	.405	1.39196	.444	1.46255	.483	1.53714
.328	1.26588	.367	1.32741	.406	1.39372	.445	1.46141	.484	1.53910
.329	1.26740	.368	1.32905	.407	1.39548	.446	1.46628	.485	1.54106
.330	1.26892	.369	1.33069	.408	1.39724	.447	1.46815	.486	1.54302
.331	1.27044	.370	1.33234	.409	1.39900	.448	1.47002	.487	1.54499
.332	1.27196	.371	1.33399	.410	1.40077	.449	1.47189	.488	1.54696
.333	1.27349	.372	1.33564	.411	1.40254	.450	1.47377	.489	1.54893
.334	1.27502	.373	1.33730	.412	1.40432	.451	1.47565	.490	1.55091
.335	1.27656	.374	1.33896	.413	1.40610	.452	1.47753	.491	1.55289
.336	1.27810	.375	1.34063	.414	1.40788	.453	1.47942	.492	1.55487
.337	1.27964	.376	1.34229	.415	1.40966	.454	1.48131	.493	1.55685
.338	1.28118	.377	1.34396	.416	1.41145	.455	1.48320	.494	1.55884
.339	1.28273	.378	1.34563	.417	1.41324	.456	1.48509	.495	1.56083
.340	1.28428	.379	1.34731	.418	1.41503	.457	1.48699	.496	1.56282
.341	1.28583	.380	1.34899	.419	1.41682	.458	1.48889	.497	1.56481
.342	1.28739	.381	1.35068	.420	1.41861	.459	1.49079	.498	1.56681
.343	1.28895	.382	1.35237	.421	1.42041	.460	1.49269	.499	1.56831
.344	1.29052	.333	1.35406	.422	1.42221	.461	1.49460	.500	1.57080

Table of Lengths of Circular Arcs whose Radius is 1.

RULE.—Knowing the measure of the circle and the measure of the arc in degrees, minutes, and seconds; take from the table the lengths opposite the number of degrees, minutes, and seconds in the arc, and multiply their sum by the radius of the circle.

EXAMPLE.—What is the length of an arc subtending an angle of 13° 27′ 8″, with a radius of 8 feet.

$$
\begin{array}{rl}
\textit{Ans.}\ \text{Length for } 13^\circ = & 0.2268928 \\
27' = & 0.0078540 \\
8'' = & 0.0000388 \\
\hline
13^\circ\ 27'\ 8'' = & 0.2347856 \\
 & 8 \\
\hline
\text{Length of arc} = & 1.8782848 \text{ feet.}
\end{array}
$$

Lengths of Circular Arcs; Radius = 1.

Sec.	Length.	Min.	Length.	Deg.	Length.	Deg.	Length.
1	0.0000048	1	0.0002909	1	0.0174533	61	1.0646508
2	0.0000097	2	0.0005818	2	0.0349066	62	1.0821041
3	0.0000145	3	0.0008727	3	0.0523599	63	1.0995574
4	0.0000194	4	0.0011636	4	0.0698132	64	1.1170107
5	0.0000242	5	0.0014544	5	0.0872665	65	1.1344640
6	0.0000291	6	0.0017453	6	0.1047198	66	1.1519173
7	0.0000339	7	0.0020362	7	0.1221730	67	1.1693706
8	0.0000388	8	0.0023271	8	0.1396263	68	1.1868239
9	0.0000436	9	0.0026180	9	0.1570796	69	1.2042772
10	0.0000485	10	0.0029089	10	0.1745329	70	1.2217305
11	0.0000533	11	0.0031998	11	0.1919862	71	1.2391838
12	0.0000582	12	0.0034907	12	0.2094395	72	1.2566371
13	0.0000630	13	0.0037815	13	0.2268928	73	1.2740904
14	0.0000679	14	0.0040724	14	0.2443461	74	1.2915436
15	0.0000727	15	0.0043633	15	0.2617994	75	1.3089969
16	0.0000776	16	0.0046542	16	0.2792527	76	1.3264502
17	0.0000824	17	0.0049451	17	0.2967060	77	1.3439035
18	0.0000873	18	0.0052360	18	0.3141593	78	1.3613568
19	0.0000921	19	0.0055269	19	0.3316126	79	1.3788101
20	0.0000970	20	0.0058178	20	0.3490659	80	1.3962634
21	0.0001018	21	0.0061087	21	0.3665191	81	1.4137167
22	0.0001067	22	0.0063995	22	0.3839724	82	1.4311700
23	0.0001115	23	0.0066904	23	0.4014257	83	1.4486233
24	0.0001164	24	0.0069813	24	0.4188790	84	1.4660766
25	0.0001212	25	0.0072722	25	0.4363323	85	1.4835299
26	0.0001261	26	0.0075631	26	0.4537856	86	1.5009832
27	0.0001309	27	0.0078540	27	0.4712389	87	1.5184364
28	0.0001357	28	0.0081449	28	0.4886922	88	1.5358897
29	0.0001406	29	0.0084358	29	0.5061455	89	1.5533430
30	0.0001454	30	0.0087266	30	0.5235988	90	1.5707963
31	0.0001503	31	0.0090175	31	0 5410521	91	1.5882496
32	0.0001551	32	0.0093084	32	0.5585054	92	1.6057029
33	0.0001600	33	0.0095993	33	0.5759587	93	1.6231562
34	0.0001648	34	0.0098902	34	0.5934119	94	1.6406095
35	0.0001697	35	0.0101811	35	0.6108652	95	1.6580628
36	0.0001745	36	0.0104720	36	0.6283185	96	1.6755161
37	0.0001794	37	0.0107629	37	0.6457718	97	1.6929694
38	0.0001842	38	0.0110538	38	0.6632251	98	1.7104227
39	0.0001891	39	0.0113446	39	0.6806784	99	1.7278760
40	0.0001939	40	0.0116355	40	0.6981317	100	1.7453293
41	0.0001988	41	0.0119264	41	0.7155850	101	1.7627825
42	0.0002036	42	0.0122173	42	0.7330383	102	1.7802358
43	0.0002085	43	0.0125082	43	0.7504916	103	1.7976891
44	0.0002133	44	0.0127991	44	0.7679449	104	1.8151424
45	0.0002182	45	0.0130900	45	0.7853982	105	1 8325957
46	0.0002230	46	0.0133809	46	0.8028515	106	1.8500490
47	0.0002279	47	0.0136717	47	0.8203047	107	1.8675023
48	0.0002327	48	0.0139626	48	0.8377580	108	1.8849556
49	0.0002376	49	0.0142535	49	0.8552113	109	1.9024089
50	0.0002424	50	0.0145444	50	0.8726646	110	1.9198622
51	0.0002473	51	0.0148353	51	0.8901179	111	1.9373155
52	0.0002521	52	0.0151262	52	0.9075712	112	1.9547688
53	0.0002570	53	0.0154171	53	0.9250245	113	1.9722221
54	0.0002618	54	0.0157080	54	0.9424778	114	1.9896753
55	0.0002666	55	0.0159989	55	0.9599311	115	2.0071286
56	0.0002715	56	0.0162897	56	0.9773844	116	2.0245819
57	0.0002763	57	0.0165806	57	0.9948377	117	2.0420352
58	0.0002812	58	0.0168715	58	1.0122910	118	2.0594885
59	0.0002860	59	0.0171624	59	1.0297443	119	2.0769418
60	0.0002909	60	0.0174533	60	1.0471976	120	2.0943951

To compute the chord of an arc when the chord of half the arc and the versed sine are given. (The versed sine is the perpendicular *bo*, Fig. 31.)

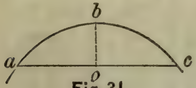


Fig. 31

RULE.—From the square of the chord of half the arc subtract the square of the versed sine, and take twice the square root of the remainder.

EXAMPLE.—The chord of half the arc is 60, and the versed sine 36, what is the length of the chord of the arc?

Ans. $60^2-36^2=2304$, and $\sqrt{2304}=48$, and $48\times2=96$, the chord.

To compute the chord of an arc when the diameter and versed sine are given.

Multiply the versed sine by 2, and subtract the product from the diameter; then subtract the square of the remainder from the square of the diameter, and take the square root of that remainder.

EXAMPLE.—The diameter of a circle is 100, and the versed sine of an arc 36, what is the chord of the arc?

Ans. $36\times2=72$. $100-72=28$. $100^2-28^2=9216$. $\sqrt{9216}=96$, the chord of the arc.

To compute the chord of half an arc when the chord of the arc and the versed sine are given.

RULE.—Take the square root of the sum of the squares of the versed sine and of half the chord of the arc.

EXAMPLE.—The chord of an arc is 96, and the versed sine 36, what is the chord of half the arc?

Ans. $\sqrt{36^2+48^2}=60$.

To compute the chord of half an arc when the diameter and versed sine are given.

RULE—Multiply the diameter by the versed sine, and take the square root of their product.

To compute a diameter.

RULE 1.—Divide the square of the chord of half the arc by the versed sine.

RULE 2.—Add the square of half the chord of the arc to the square of the versed sine, and divide this sum by the versed sine.

EXAMPLE.—What is the radius of an arc whose chord is 96, and whose versed sine is 36?

Ans. $48^2+36^2=3600$. $3600\div36=100$, the diameter, and radius$=50$.

To compute the versed sine.

RULE.—Divide the square of the chord of half the arc by the diameter.

To compute the versed sine when the chord of the arc and the diameter are given.

RULE.—From the square of the diameter subtract the square of the chord, and extract the square root of the remainder; subtract this root from the diameter, and halve the remainder.

To compute the length of an arc of a circle when the number of degrees and the radius are given.

RULE 1.—Multiply the number of degrees in the arc by 3.1416 multiplied by the radius, and divide by 180. The result will be the length of the arc in the same unit as the radius.

RULE 2.—Multiply the radius of the circle by 0.01745, and the product by the degrees in the arc.

EXAMPLE.—The number of degrees in an arc is 60, and the radius is 10 inches, what is the length of the arc in inches?

Ans. $10\times3.1416\times60=1884.96\div180=10.47$ inches; or, $10\times0.01745\times60=10.47$ inches.

To compute the length of the arc of a circle when the length is given in degrees, minutes, and seconds.

RULE 1.—Multiply the number of degrees by 0.01745329, and the product by the radius.

RULE 2.—Multiply the number of minutes by 0.00029, and that product by the radius.

RULE 3.—Multiply the number of seconds by 0.0000048 times the radius. Add together these three results for the length of the arc.

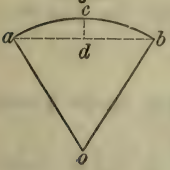
See also table, p. 59.

EXAMPLE.—What is the length of an arc of $60°\ 10'\ 5''$, the radius being 4 feet?

$$\begin{array}{lllr} \textit{Ans.}\ 1. & 60°\times0.01745329 & \times4= & 4.188789\ \text{feet.} \\ 2. & 10'\times0.00029 & \times4= & 0.0116\ \text{feet.} \\ 3. & 5''\times0.0000048 & \times4= & 0.000096\ \text{feet.} \\ \hline & & & 4.200485\ \text{feet.} \end{array}$$

To compute the area of a sector of a circle when the degrees of the arc and the radius are given (Fig. 32).*

Fig. 32



RULE.—Multiply the number of degrees in the arc by the area of the whole circle, and divide by 360.

EXAMPLE.—What is the area of a sector of a circle whose radius is 5, and the length of the arc is 60°?

Ans. Area of circle $=10\times10\times0.7854=78.54$.

$$\text{Then area of sector}=\frac{78.5\times60}{360}=13.09.$$

If the length of the arc is given in degrees and minutes, reduce it to minutes, and multiply by the area of the whole circle, and divide by 21600.

To compute the area of a sector of a circle when the length of the arc and radius are given.

RULE.—Multiply the length of the arc by half the length of the radius, and the product is the area.

To compute the area of a segment of a circle when the chord and versed sine of the arc and the radius or diameter of the circle are given.

NOTE.—The versed sine is the distance *cd* (Fig. 32).

RULE 1 (*when the segment is less than a semicircle*).—Ascertain the area of the sector having the same arc as the segment, then ascertain the area of a triangle formed by the chord of the segment and the radii of the sector, and take the difference of these areas.

RULE 2 (*when the segment is greater than a semircicle*).—Ascertain by the preceding rule the area of the lesser portion of the circle, subtract it from the area of the whole circle, and the remainder will give the area.

To compute the convex surface of a sphere.

RULE.—Multiply the diameter by the circumference, and the product will give the surface.

EXAMPLE.—What is the convex surface of a sphere of 10 inches diameter?

Ans. Circumference of sphere $=10\times3.1416=31.416$ inches; $10\times31.416=314.16$ sq. in., the surface of sphere.

* The degrees of the arc are the same as of the angle *aob*.

To compute the surface of a segment of a sphere.

RULE.—Multiply the height (bc, Fig. 33) by the circumference of the sphere, and add the product to the area of the base.

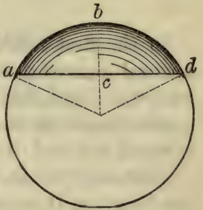


Fig. 33

To find the area of the base, we have the diameter of the sphere and the length of the versed sine of the arc abd, and we can find the length of the chord ad by the rule on p. 60. Having, then, the length of the chord ad for the diameter of the base, we can easily find the area.

EXAMPLE.—The height, bc, of a segment abd, is 36 inches, and the diameter of the sphere is 100 inches. What is the convex surface, and what the whole surface?

Ans. $100 \times 3.1416 = 314.16$ inches, the circumference of sphere.

$36 \times 314.16 = 11309.76$, the convex surface.

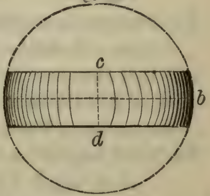
The length of $ad = 100 - \overline{36 \times 2} = 28$.

$\sqrt{100^2 - 28^2} = 96$, the chord ad.

$96^2 \times 0.7854 = 2738.2464$, the area of base.

$11309.76 + 7238.2464 = 18548.0064$, the total area.

Fig. 34



To compute the surface of a spherical zone.

RULE.—Multiply the height (cd, Fig. 34) by the circumference of the sphere for the convex surface, and add to it the area of the two ends for the whole area.

Spheroids, or Ellipsoids.

DEFINITION.—Spheroids, or ellipsoids, are figures generated by the revolution of a semi-ellipse about one of its diameters.

When the revolution is about the long diameter, they are *prolate;* and when it is about the short diameter, they are *oblate.*

A prolate spheroid is cigar-shaped, an oblate spheroid is like a watch.

To compute the surface of a spheroid.

Let $a = \frac{1}{2}$ long axis; let $b = \frac{1}{2}$ short axis;

let $$\frac{a^2 - b^2}{a^2} = e^2 \quad \text{or} \quad e = \sqrt{\frac{a^2 - b^2}{a^2}};$$

then *surface of oblate spheroid*

$$2 = \pi a^2 + \frac{\pi b^2}{e} \log\left(\frac{1+e}{1-e}\right);$$

Surface of *prolate spheroid*

$$=2\pi b^2+2\pi ab\frac{sin^{-1}e}{e}.$$

In the first formula, *natural* logarithms must be used. The natural logarithm may be obtained by multiplying the common logarithm by 2.302.

$sin^{-1}e$ may be found, by finding the angle whose natural sine is equal to e and dividing the angle so obtained by 57.3.

[Although the above formulæ are complicated, no simpler rules can be given that are at all reliable.]

To compute the surface of a cylinder.

RULE.—Multiply the length by the circumference for the convex surface, and add to the product the area of the two ends for the whole surface.

To compute the sectional area of a circular ring (Fig. 35).

RULE.—Find the area of both circles, and subtract the area of the smaller from the area of the larger; the remainder will be the area of the ring.

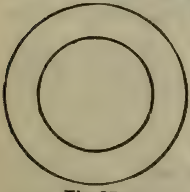


Fig. 35

To compute the surface of a cone.

RULE.—Multiply the perimeter or circumference of the base by one-half the slant height, or side of the cone, for the convex area. Add to this the area of the base, for the whole area.

EXAMPLE.—The diameter of the base of a cone is 3 inches, and the slant height 15 inches, what is the area of the cone?

Ans. $3\times3.1416=$ 9.4248 = circumference of base.
$9.4248\times7\frac{1}{2}=$ 70.686 square inches, the convex surface.
$3\times3\times0.7854=$ 7.068 square inches, the area of base.
Area of cone = 77.754 square inches.

To compute the area of the surface of the frustum of a cone.

RULE.—Multiply the sum of the perimeters of the two ends by the slant height of the frustum, and divide by 2, for the convex surface. Add the area of the top and bottom surfaces.

To compute the surface of a pyramid.

RULE.—Multiply the perimeter of the base by one-half the slant height, and add to the product the area of the base.

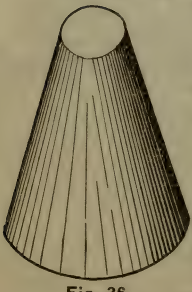


Fig. 36

To compute the surface of the frustum of a pyramid.

RULE.—Multiply the sum of the perimeters of the two ends by the slant height of the frustum, halve the product, and add to the result the area of the two ends.

MENSURATION OF SOLIDS.

To compute the volume of a prism.

RULE.—Multiply the area of the base by the height.

This rule applies to any prism of any shape on the base, as long as the top and bottom surfaces are parallel.

To compute the volume of a prismoid.

DEFINITION.—A prismoid is a solid having parallel ends or bases dissimilar in shape with quadrilateral sides.

RULE.—To the sum of the areas of the two ends add four times the area of the middle section parallel to them, and multiply this sum by one-sixth of the perpendicular height.

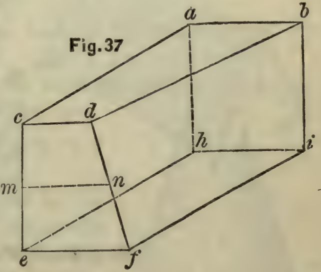


Fig. 37

EXAMPLE.—What is the volume of a quadrangular prismoid, as in Fig. 37, in which $ab=6''$, $cd=4''$, $ac=he=10''$, $ce=8''$, $ef=8''$, and $ih=6''$?

Ans. Area of top $=\frac{6+4}{2}\times 10=50.$

Area of bottom $=\frac{8+6}{2}\times 10=70.$

Area of middle section $=\frac{6+6}{2}\times 10=60.$

$[50+70+(4\times 60)]\times\frac{8}{6}=480$ cubic inches.

NOTE—The length of the end of the middle section, as mn in Fig. 37 $=\frac{ed+ef}{2}$.

To find the volume of a prism truncated obliquely.

RULE.—Multiply the area of the base by the average height of the edges.

EXAMPLE.—What is the volume of a truncated prism, as in Fig. 38, where $ef=6$ inches, $fh=10$ inches, $ea=10$, $ci=12$, $dh=8$, and $fb=8$?

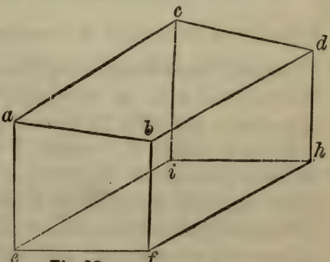


Fig. 38

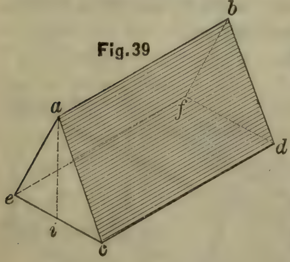
Ans. Area of base $=6\times10 \quad =60$ square inches.

Average height of edges $=\dfrac{10+12+8+8}{4}=9\frac{1}{2}$ inches.

$60\times9\frac{1}{2}=970$ cubic inches.

To compute the volume of a wedge when the ends are parallel and equal.

Fig. 39



RULE.—Multiply the area of one end by the length of the wedge.

To compute the volume of a wedge when the ends are not parallel.

RULE.—Add together the lengths of the three edges, *ab*, *cd*, and *ef*; multiply their sum by the perpendicular height of the wedge, and then by the breadth of the back, and divide the product by 6.

Regular Polyhedrons.

DEFINITION.—A regular body is a solid contained within a certain number of similar and equal plane faces, all of which are equal regular polygons.

The whole number of regular bodies which can possibly be found is five. They are:—

1. The *tetrahedron*, or pyramid.
2. The *hexahedron*, or cube, which has six square faces.
3. The *octahedron*, which has eight triangular faces.
4. The *dodecahedron*, which has twelve pentagonal faces.
5. The *icosahedron*, which has twenty triangular faces.

To compute the volume of a regular polyhedron.

RULE 1 (*when the radius of the circumscribing sphere is given*).—Multiply the cube of the radius of the sphere by the multiplier opposite to the body in column 2 of the following table.

RULE 2 (*when the radius of the inscribed sphere is given*).—Multiply the cube of the radius of the inscribed sphere by the multiplier opposite to the body in column 3 of the following table.

RULE 3 (*when the surface is given*).—Cube the surface given, extract the square root, and multiply the root by the multiplier opposite to the body in column 4 of the following table.

Figure.	1 No. of sides.	2 Volume by radius of circumscribing sphere.	3 Volume by radius of inscribed circle.	4 Volume by surface.
Tetrahedron...........	4	0.5132	13.85641	0.0517
Hexahedron...........	6	1.5396	8.0000	0.06804
Octahedron...........	8	1.33333	6.9282	0.07311
Dodecahedron.........	12	2.78517	5.55029	0.08169
Icosahedron...........	20	2.53615	5.05406	0.0856

To compute the volume of a cylinder.

RULE.—Multiply the area of the base by the height.

To compute the volume of a cone.

RULE.—Multiply the area of the base by the perpendicular height, and take one-third of the product.

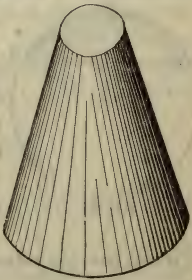


Fig. 40

To compute the volume of the frustum of a cone.

RULE.—Add together the squares of the diameters of the two ends and the product of the two diameters; multiply this sum by 0.7854, and this product by the height, and then divide this last product by 3.

EXAMPLE —What is the volume of a frustum of a cone 9 inches high, 5 inches diameter at the base, and 3 inches at the top?

Ans. $5^2+3^2=34$. $3\times5=15$. $15+34=49$, the sum of the squares and product of the diameters. $49\times0.7854=38.4846$.

$$\frac{38{,}4846\times9}{3}=115.4538 \text{ cubic inches.}$$

To compute the volume of a pyramid.

RULE.—Multiply the area of the base by the perpendicular height, and take one-third of the product.

To compute the volume of the frustum of a pyramid.

RULE.—Find the height that the pyramid would be if the top were put on, and then compute the volume of the completed pyramid and the volume of the part added; subtract the latter from the former, and the remainder will be the volume of the frustum.

To compute the volume of a sphere.

RULE.—Multiply the cube of the diameter by 0.5236.

To compute the volume of a segment of a sphere.

RULE 1.—To three times the square of the radius of its base add the square of its height; multiply this sum by the height, and the product by 0.5236.

RULE 2.—From three times the diameter of the sphere subtract twice the height of the segment; multiply this remainder by the square of the height, and the product by 0.5236.

EXAMPLE.—The segment of a sphere has a radius, *ac* (Fig. 41), of 7 inches for its base, and a height, *cb*, of 4 inches: what is its volume?

Ans. (by Rule 1). $3 \times 7^2 = 147$, and $147 + 4^2 = 163$, three times the square of the radius of the base plus the square of the height. $163 \times 4 \times 0.5236 = 341.3872$ cubic inches volume.

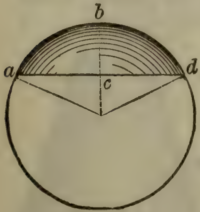


Fig 41.

SECOND SOLUTION.—By the rule for finding the diameter of a circle when a chord and its versed sine are given, we find that the diameter of the sphere in this case is 16.25 inches; then, by Rule 2, $(3 \times 16.25) - (2 \times 4) = 40.75$, and $40.75 \times 4^2 \times 0.5236 = 341.3872$ cubic inches, the volume of the segment.

To compute the volume of a spherical zone.

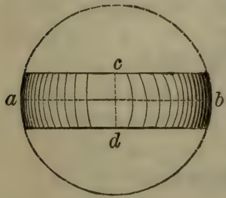


Fig. 42

DEFINITION.—The part of a sphere included between two parallel planes (Fig. 42).

RULE.—To the sum of the squares of the radii of the two ends add one-third of the square of the height of the zone; multiply this sum by the height, and that product by 1.5708.

To compute the volume of a prolate spheroid (see page 63).

Fig.43

RULE.—Multiply the square of the short axis by the long axis, and this product by 0.5236.

To compute the volume of an oblate spheroid.

RULE.—Multiply the square of the long axis by the short axis, and this product by 0.5236.

To compute the volume of a paraboloid of revolution (Fig. 43).

RULE.—Multiply the area of the base by half the altitude.

To compute the volume of a hyperboloid of revolution (Fig. 44).

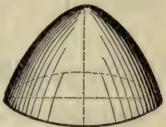


Fig.44

Rule.—To the square of the radius of the base add the square of the middle diameter; multiply this sum by the height, and the product by 0.5236.

To compute the volume of any figure of revolution.

Rule.—Multiply the area of the generating surface by the circumference described by its centre of gravity.

To compute the volume of an excavation, where the ground is irregular, and the bottom of the excavation is level (Fig. 45).

Rule.—Divide the surface of the ground to be excavated into equal squares of about 10 feet on a side, and ascertain by means of a level the height of each corner, *a, a, a, b, b, b,* etc., above the level to which the ground is to be excavated. Then add together the heights of all the corners that only come into one square. Next take twice the sum of the heights of all the corners that come in two squares, as *b, b, b*; next three times the sum of the heights of all the corners that come in three squares, as *c, c, c*; and then four times the sum of the heights of all the corners that belong to four squares, as *d, d, d,* etc. Add together all these quantities, and multiply their sum by one-fourth the area of one of the squares. The result will be the volume of the excavation.

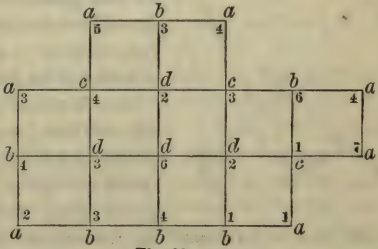


Fig.45

Example.—Let the plan of the excavation for a cellar be as in the figure, and the heights of each corner above the proposed bottom of the cellar be as given by the numbers in the figure, then the volume of the cellar would be as follows, the area of each square being $10 \times 10 = 100$ square feet:—

Volume $= \frac{1}{4}$ of 100 (a's $+ 2\ b$'s $+ 3\ c$'s $+ 4\ d$'s).

The a's in this case $= 4+6+3+2+1+7+4 = 27$

$2\times$ the sum of the b's $= 2 \times (3+6+1+4+3+4) = 42$

$3\times$ the sum of the c's $= 3 \times (1+3+4) = 24$

$4\times$ the sum of the d's $= 4 \times (2+3+6+2) = 52$

145

Volume $=25\times145=3625$ cubic feet, the quantity of earth to be excavated.

GEOMETRICAL PROBLEMS.

PROBLEM 1.—*To bisect, or divide into equal parts, a given line, ab* (Fig. 46).

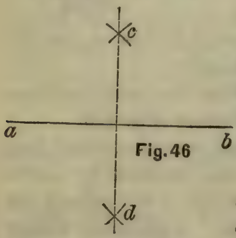


Fig. 46

From *a* and *b*, with any radius greater than half of *ab*, describe arcs intersecting in *c* and *d*. The line *cd*, connecting these intersections, will bisect *ab*, and be perpendicular to it.

PROBLEM 2.—*To draw a perpendicular to a given straight line from a point without it.*

1ST METHOD (Fig. 47).—From the point *a* describe an arc with sufficient radius that it will cut the line *bc* in two places, as *e* and *f*. From *e* and *f* describe two arcs, with the same radius, intersecting in *g;* then a line drawn from *a* to *g* will be perpendicular to the line *bc*.

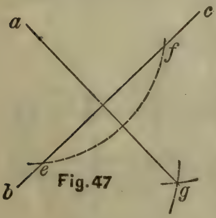


Fig. 47

2D METHOD (Fig. 48).—From any two points, *d* and *c*, at some distance apart in the given line, and with radii *da* and *ca* respectively, describe arcs cutting at *a* and *e*. Draw *ae*, and it will be the perpendicular required. This method is useful where the given point is opposite the end of the line, or nearly so.

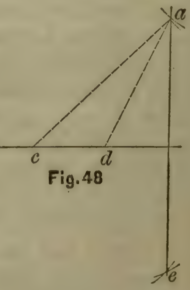


Fig. 48

PROBLEM 3.—*To draw a perpendicular to a straight line from a given point, a, in that line.*

1ST METHOD (Fig. 49).—With any radius, from the given point *a* in the line, describe arcs cutting the line in the points *b* and *c*. Then with *b* and *c* as centres, and with any radius greater than *ab* or *ac*, describe arcs cutting each other at *d*. The line *da* will be the perpendicular desired.

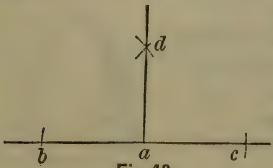


Fig. 49

2D METHOD (Fig. 50, when the given point is at the end of the line).—From any point, b, outside of the line, and with a radius ba, describe a semicircle passing through a, and cutting the given line at d. Through b and d draw a straight line intersecting the semicircle at e. The line ea will then be perpendicular to the line ac at the point a.

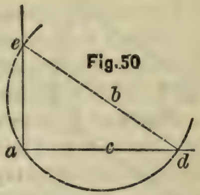


Fig. 50

3D METHOD (Fig. 51) OR THE 3, 4, AND 5 METHOD.—From the point a on the given line measure off 4 inches, or 4 feet, or 4 of any other unit, and with the same unit of measure describe an arc, with a as a centre and 3 units as a radius. Then from b describe an arc, with a radius of 5 units, cutting the first arc in c. Then ca will be the perpendicular. This method is particularly useful in laying out a right angle on the ground, or framing a house where the foot is used as the unit, and the lines laid off by straight edges.

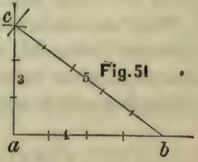


Fig. 51

In laying out a right angle on the ground, the proportions of the triangle may be 30, 40, and 50, or any other multiple of 3, 4, and 5; and it can best be laid out with the tape. Thus, first measure off, say 40 feet from a on the given line, then let one person hold the end of the tape at b, another hold the tape at the 80-foot mark at a, and a third person take hold of the tape at the 50-foot mark, with his thumb and finger, and pull the tape taut. The 50-foot mark will then be at the point c in the line of the perpendicular.

PROBLEM 4.—*To draw a straight line parallel to a given line at a given distance apart* (Fig. 52).

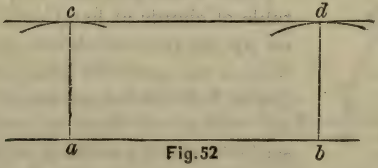


Fig. 52

From any two points near the ends of the given line describe two arcs about opposite the line. Draw the line cd tangent to these arcs, and it will be parallel to ab.

PROBLEM 5.—*To construct an angle equal to a given angle.*

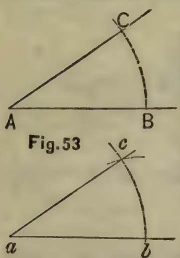


Fig. 53

With the point A, at the apex of the given angle, as a centre, and any radius, describe the arc BC. Then with the point a, at the vertex of the new angle, as a centre, and with the same radius as before, describe an arc like BC. Then with BC as a radius, and b as a centre, describe an arc cutting the other at c. Then will cab be equal to the given angle CAB.

PROBLEM 6.—*From a point on a given line to draw a line making an angle of* 60° *with the given line* (Fig. 54).

Take any distance, as ab, as a radius, and, with a as a centre, describe the arc bc. Then with b as a centre, and the same radius, describe an arc cutting the first one at c. Draw from a a line through c, and it will make with ab an angle of 60°.

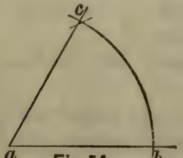


Fig. 54

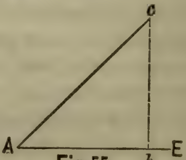


Fig. 55

PROBLEM 7.—*From a given point, A, on a given line, AE, to draw a line making an angle of* 45° *with the given line* (Fig. 55).

Measure off from A, on AE, any distance, Ab, and at b draw a line perpendicular to AE. Measure off on this perpendicular bc equal to Ab, and draw a line from A through c, and it will make an angle with AE of 45°.

PROBLEM 8.—*From any point, A, on a given line, to draw a line which shall make any desired angle with the given line* (Fig. 56).

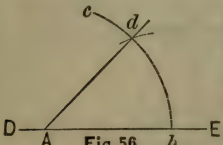


Fig. 56

To perform this problem we must have a table of chords at hand (such as is found on pp. 88–96), which we use as follows. Find in the table the length of chord to a radius 1, for the given angle. Then take any radius, as large as convenient, describe an arc of a circle bc with A as a centre. Multiply the chord of the angle, found in the table, by the length of the radius Ab, and with the product as a new radius, and b as a centre, describe a short arc cutting bc in d. Draw a line from A through d, and it will make the desired angle with DE.

Example.—Draw a line from A on DE, making an angle of 44° 40′ with DE.

Solution.—We find that the largest convenient radius for our arc is 8 inches: so with A as a centre, and 8 inches as a radius, we describe the arc bc. Then, looking in the table of chords, we find the chord for an angle or arc of 44° 40′ to a radius 1 is 0.76. Multiplying this by 8 inches, we have, for the length of our new radius, 6.08 inches, and with this as radius, and b as a centre, we describe an arc cutting bc in d. Ad will then be the line desired.

Problem 8*a*.—*To lay off a given angle approximately by means of an ordinary two-foot rule.* Lay one leg of the rule on the paper or board with its inner edge coinciding with the given line. Open the rule until the distance between the inner edges at the ends corresponds with that given for the angle in the following table; then draw a line by marking along the inner edge of the other leg, and it will give the desired angle within a very close approximation.

TABLES OF ANGLES CORRESPONDING TO OPENINGS OF A TWO-FOOT RULE (Trautwine).

In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.
¼	1	12		11	22	4½	21	37		32	3	8¾	42	46
	1	48	2½	11	58		22	13	6¾	32	40		43	24
½	2	24		12	34	¾	22	50		33	17	9	44	3
	3	00	¾	13	10		23	27	7	33	54		44	42
¾	3	36		13	46	5	24	3		34	33	¼	45	21
	4	11	3	14	22		24	39	¼	35	10		45	59
1	4	47		14	58	¼	25	16		35	47	½	46	38
	5	23	¼	15	34		25	53	½	36	25		47	17
¼	5	58		16	10	½	26	30		37	3	¾	47	56
	6	34	½	16	46		27	7	¾	37	41		48	35
½	7	10		17	22	¾	27	44		38	19	10	49	15
	7	46	¾	17	59		28	21	8	38	57		49	54
¾	8	22		18	35	6	28	58		39	35	¼	50	34
	8	58	4	19	12		29	35	¼	40	13		51	13
2	9	34		19	48	¼	30	11		40	51	½	51	53
	10	10	¼	20	24		30	49	½	41	29		52	33
¼	10	46		21	00	½	31	26		42	7			

Problem 9.—*To bisect a given angle, as BAC* (Fig. 57).

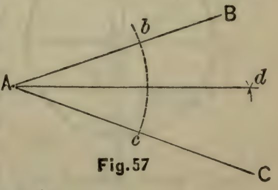


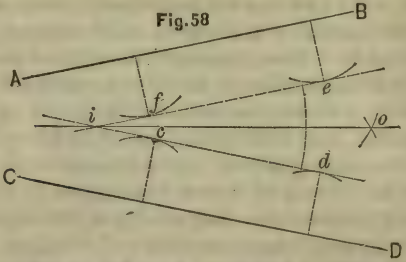
Fig. 57

With A as a centre, and any radius, describe an arc, as cb. With c and b as centres, and any radius greater than one-half of cb, describe two arcs intersecting in d. Draw from A a line through d, and it will bisect the angle BAC.

Problem 10.—*To bisect the angle contained between two lines,*

as AB and CD, when the vertex of the angle is not on the drawing (Fig. 58).

Fig. 58



Draw *fe* parallel to *AB*, and *cd* parallel to *CD*, so that the two lines will intersect each other, as at *i*. Bisect the angle *eid*, as in the preceding problem, and draw a line through *i* and *o* which will bisect the angle between the two given lines.

PROBLEM 11.—*Through two given points, B and C, to describe an arc of a circle with a given radius* (Fig. 59).

With *B* and *C* as centres, and a radius equal to the given radius, describe two arcs intersecting at *A*. With *A* as a centre, and the same radius, describe the arc *bc*, which will be found to pass through the given points, *B* and *C*.

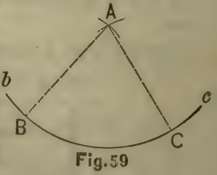


Fig. 59

PROBLEM 12.—*To find the centre of a given circle* (Fig. 60).

Draw any chord in the circle, as *ab*, and bisect this chord by the perpendicular *cd*. This line will pass through the centre of the circle, and *ef* will be a diameter of the circle. Bisect *ef*, and the centre *o* will be the centre of the circle.

Fig. 60

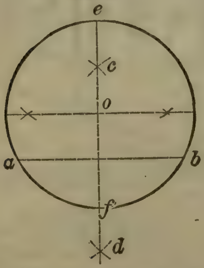
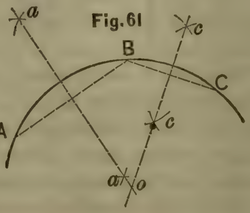


Fig. 61



PROBLEM 13.—*To draw a circular arc through three given points, as A, B, and C* (Fig. 61).

Draw a line from A to B and from B to C. Bisect AB and BC by the lines aa and cc, and prolong these lines until they intersect at o, which will be the centre for the arc sought. With o as a centre, and Ao as a radius, describe the arc ABC.

PROBLEM 14.—*To describe a circular arc passing through three given points, when the centre is not available, by means of a triangle* (Fig. 62).

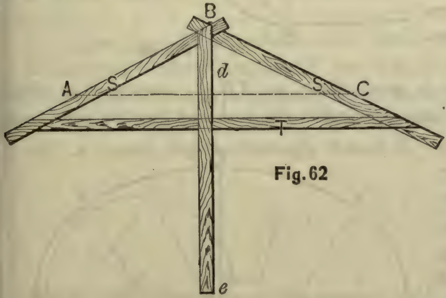


Fig. 62

Let A, B, and C be the given points. Insert two stiff pins or nails at A and C. Place two strips of wood, SS, as shown in the figure; one against A, the other against C, and inclined so that their intersection shall come at the third point, B. Fasten the strips together at their intersection, and nail a third strip, T, to their other ends, so as to make a firm triangle. Place the pencil-point at B, and, keeping the edges of the triangle against A and B, move the triangle to the left and right, and the pencil will describe the arc sought.

When the points A and C are at the same distance from B, if a strip of wood be nailed to the triangle, so that its edge de shall be at right angles to a line joining A and C as the triangle is moved one way or the other, the edge de will always point to the centre of the circle. This principle is used in the perspective lineard.

PROBLEM 15.—*To find a circular arc which shall be tangent to a given point, A, on a straight line, and pass through a given point, C, outside the line* (Fig. 63).

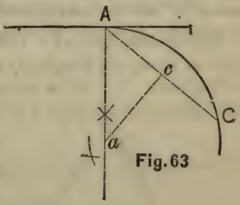


Fig. 63

Draw from A a line perpendicular to the given line. Connect A and C by a straight line, and bisect it by the perpendicular ac. The point where these two perpendiculars intersect will be the centre of the circle.

PROBLEM 16.—*To connect two parallel lines by a reversed curve composed of two circular arcs of equal radius, and tangent to the lines at given points, as A and B* (Fig. 64).

Join A and B, and divide the line into two equal parts at C. Bisect CA and CB by perpendiculars. At A and B erect perpendiculars to the given lines, and the intersections a and b will be the centres of the arcs composing the required curve.

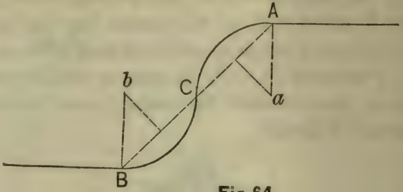


Fig. 64

PROBLEM 17.—*On a given line, as AB, to construct a compound curve of three arcs of circles, the radii of the two side ones being equal and of a given length, and their centres in the given line; the central arc to pass through a given point, C, on the perpendicular bisecting the given line, and tangent to the other two arcs* (Fig. 65).

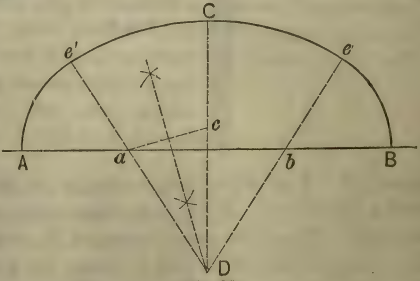


Fig. 65

Draw the perpendicular CD. Lay off Aa, Bb, and Cc, each equal to the given radius of the side arcs; join ac; bisect ac by a perpendicular. The intersection of this line with the perpendicular CD will be the required centre of the central arc. Through a and b draw the lines De and De'; from a and b, with the given radius, equal to Aa, Bb, describe the arcs Ae' and Be; from D as a centre, and CD as a radius, describe the arc eCe' which completes the curve required.

PROBLEM 18.—*To construct a triangle upon a given straight line or base, the length of the two sides being given* (Fig. 66).

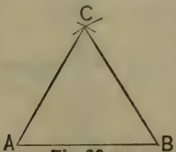


Fig. 66 a

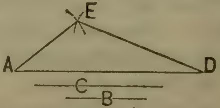


Fig. 66 b

First (an equilateral triangle, Fig. 66a).—With the extremities

A and B of the given line as centres, and AB as a radius, describe arcs cutting each other at C. Join AC and BC.

Second (when the sides are unequal, Fig. 66*b*).—Let AD be the given base, and the other two sides be equal to C and B. With D as a centre, and a radius equal to C, describe an indefinite arc. With A as a centre, and B as a radius, describe an arc cutting the first at E. Join E with A and D, and it will give the required triangle.

PROBLEM 19.—*To describe a circle about a triangle* (Fig. 67).

Bisect two of the sides, as AC and CB, of the triangle, and at their centres erect perpendicular lines, as ae and be, intersecting at e. With e as a centre, and eC as a radius, describe a circle, and it will be found to pass through A and B.

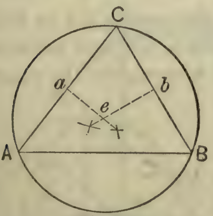


Fig. 67

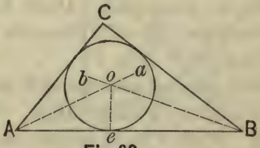


Fig. 68

PROBLEM 20.—*To inscribe a circle in a triangle* (Fig. 68).

Bisect two of the angles, A and B, of the triangle by lines cutting each other at o. With o as a centre, and oe as a radius, describe a circle, which will be found to just touch the other two sides.

PROBLEM 21.—*To inscribe a square in a circle, and to describe a circle about a square* (Fig. 69).

To inscribe the square. Draw two diameters, AB and CD, at right angles to each other. Join the points A, D, B, C, and we have the inscribed square.

To describe the circle. Draw the diagonals as before, intersecting at E, and, with E as a centre and AE as a radius, describe the circle.

PROBLEM 22.—*To inscribe a circle in a square, and to describe a square about a circle* (Fig. 70).

To inscribe the circle. Draw the diagonals AB and CD, intersecting at E. Draw the perpendicular EG to one of the sides. Then with E as a centre, and EG as a radius, describe a circle, which will be found to touch all four sides of the square.

To describe the square. Draw two diameters, AB and CD, at right angles to each other, and prolonged beyond the circumference.

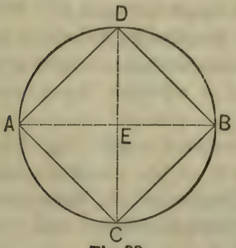


Fig. 69

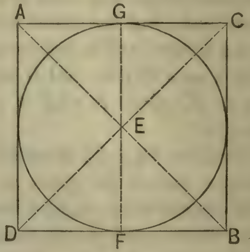


Fig. 70

ence. Draw the diameter GF, bisecting the angle CEA or BED. Draw lines through G and F perpendicular to GF, and terminating in the diagonals. Draw AD and CB to complete the square.

PROBLEM 23.—*To inscribe a pentagon in a circle* (Fig. 71).

Draw two diameters, AB and CD, at right angles to each other. Bisect AO at E. With E as a centre, and EC as a radius, cut OB at F. With C as a centre and CF as a radius, cut the circle at G and H. With these points as centres, and the same radius, cut the circle at I and J. Join I, J, H, G, and C, and we then have inscribed in the circle a regular pentagon.

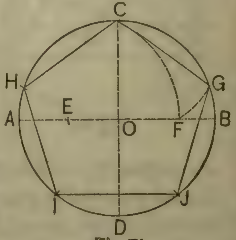


Fig. 71

PROBLEM 24.—*To inscribe a regular hexagon in a circle* (Fig. 72).

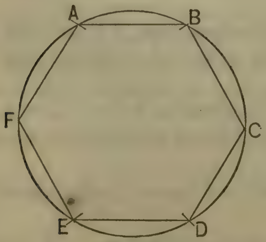


Fig. 72

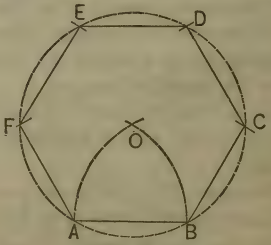


Fig. 73

SOLUTION.—Lay off on the circumference the radius of the circle six times, and connect the points.

Problem 25.—*To construct a regular hexagon upon a given straight line, AB* (Fig. 73).

From A and B, with a radius equal to AB, describe arcs cutting at O. With O as a centre, and a radius equal to AB, describe a circle, and from A and B lay off the length AB on the circumference of the circle, and join the points thus obtained. The result will be a regular hexagon.

Problem 26.—*To construct a regular octagon upon a given straight line, AB* (Fig. 74).

Produce the line AB both ways, and draw the perpendiculars Aa and Bb, of indefinite length. Bisect the external angles at A and B, and make the length of the lines equal to AB. From H and C draw lines parallel to Aa, and equal in length to AB; and

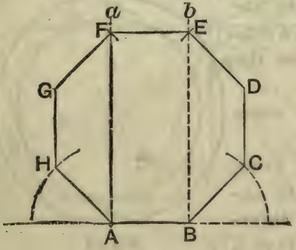


Fig. 74

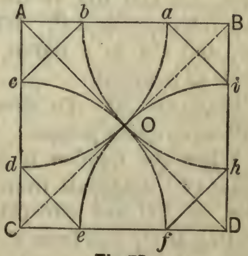


Fig. 75

from the centres G and D describe arcs, with a radius AB, cutting the perpendiculars Aa and Bb in F and E. Join GF, FE, and ED.

Problem 27.—*To make a regular octagon from a square* (Fig. 75).

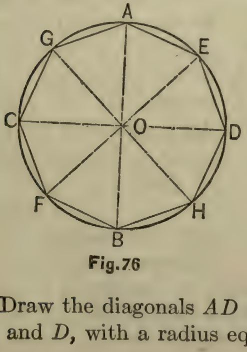


Fig. 76

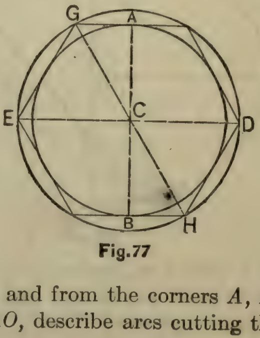


Fig. 77

Draw the diagonals AD and BC, and from the corners A, B, C, and D, with a radius equal to AO, describe arcs cutting the

sides of the square in *a*, *b*, *c*, *d*, *e*, *f*, *h*, and *i*. Join these points to complete the octagon.

PROBLEM 28.—*To inscribe a regular octagon in a circle* (Fig. 76).

Draw two diameters, *AB* and *CD*, at right angles to each other. Bisect the angles *AOD* and *AOC* by the diameters *EF* and *GH*. Join *A*, *E*, *D*, *H*, *B*, etc., for the inscribed figure.

PROBLEM 29.—*To inscribe a circle within a regular polygon.*

First (when the polygon has an even number of sides, as in Fig. 77).—Bisect two opposite sides at *A* and *B*, and draw *AB*, and bisect it at *C* by a diagonal, *DE*, drawn between two opposite angles. With the radius *CA* describe the circle as required.

Second (when the number of sides is odd, as in Fig. 78).—Bisect two of the sides at *A* and *B*, and draw lines, *AE* and *BD*, to the opposite angles, intersecting at *C*. With *C* as a centre, and *CA* as a radius, describe the circle as required.

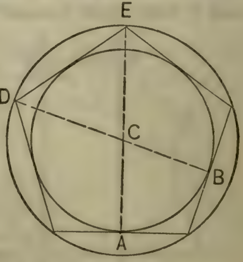


Fig. 78

PROBLEM 30.—*To describe a circle without a regular polygon.*

When the number of sides is even, draw two diagonals from opposite angles, as *ED* and *GH* (Fig. 77), intersecting at *C*; and from *C*, with *CD* as a radius, describe the circle required.

When the number of sides is odd, find the centre, *C*, as in last problem; and with *C* as a centre, and *CD* (Fig. 78) as a radius, describe the circle required.

PROBLEMS ON THE ELLIPSE, THE PARABOLA, THE HYPERBOLA, AND THE CYCLOID.

The Ellipse.

PROBLEM 31.—*To describe an ellipse, the length and breadth, or the two axes, being given.*

1ST METHOD (Fig. 79, the two axes, *AB* and *CD*, being given). — On *AB* and *CD* as diameters, and from the same centre, *O*, describe the circles *AGBH* and *CLDK*. Take any convenient number of points on the circumference of the outer circle, as *b*, *b′*, *b″*, etc., and from them draw lines to the centre, *O*, cutting the inner circle at the points *a*, *a′*, *a″*, etc., respectively. From the points *b*, *b′*, etc., draw lines parallel to the shorter axis; and from the points *a*, *a′*, etc., draw lines parallel to the longer axis, and intersecting the first set of lines at *c*, *c′*, *c″*, etc. These last points will be points in the ellipse, and, by obtaining a sufficient number of them, the ellipse can easily be drawn.

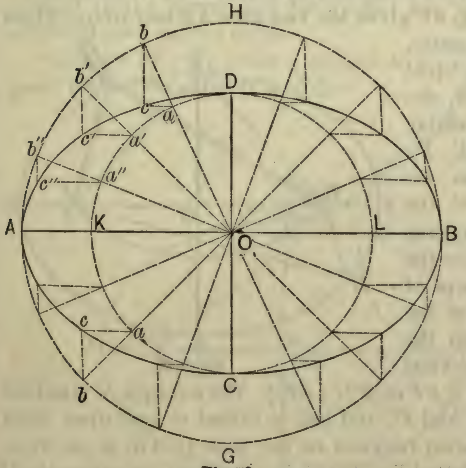


Fig.79

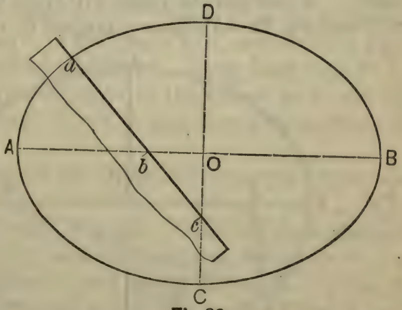


Fig.80

2D METHOD (Fig. 80). —Take the straight edge of a stiff piece of paper, cardboard, or wood, and from some point as *a*, mark off *ab* equal to half the shorter diameter, and *ac* equal to half the longer diameter. Place the straight edge so

that the point b shall be on the longer diameter, and the point c on the shorter: then will the point a be over a point in the ellipse. Make on the paper a dot at a, and move the slip around, always keeping the points b and c over the major and minor axes. In this way any number of points in the ellipse may be obtained, which may be connected by a curve drawn freehand.

3D METHOD (Fig. 81, given the two axes AB and CD).—From the point D as a centre, and a radius AO, equal to one-half of AB, describe an arc cutting AB at F and F'. These two points are called the foci of the ellipse. [One property of the ellipse is, that the sum of the distances of any two points on the circumference from the foci is the same. Thus

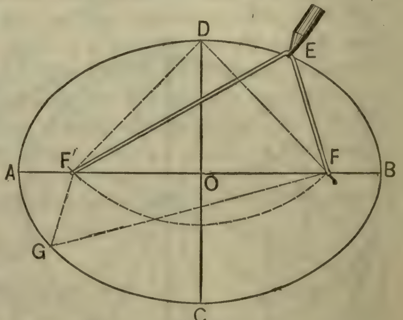


Fig. 81

$F'D + DF = F'E + EF$ or $F'G + GF$.] Fix a couple of pins into the axis AB at F and F', and loop a thread or cord upon them equal in length, when fastened to the pins, to AB, so as, when stretched as per dotted line FDF', just to reach the extremity D of the short axis. Place a pencil-point inside the chord, as at E, and move the pencil along, always keeping the cord stretched tight. In this way the pencil will trace the outline of the ellipse.

PROBLEM 32.—*To draw a tangent to an ellipse at a given point on the curve* (Fig. 82).

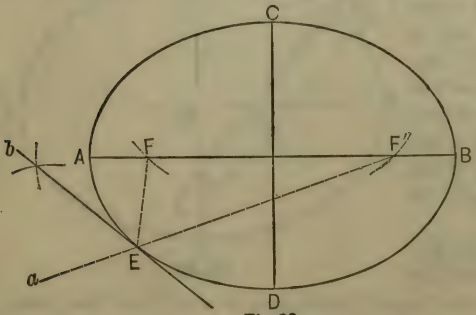


Fig. 82

Let it be required to draw a tangent at the point E on the ellipse shown in Fig. 82. First find the foci F and F', as in the third method for describing an ellipse, then from E draw lines EF and EF'. Prolong EF' to a, so that Ea shall

equal EF. Bisect the angle aEF as at b, and through b draw a line touching the curve at E. This line will be the tangent required. If it were desired to draw a line normal to the curve at E, as, for instance, the joint of an elliptical arch, bisect the angle FEF', and draw the bisecting line through E, and it will be the normal to the curve, and the proper line for the joint of an elliptical arch at that point.

PROBLEM 33.—*To draw a tangent to an ellipse from a given point without the curve* (Fig. 83).

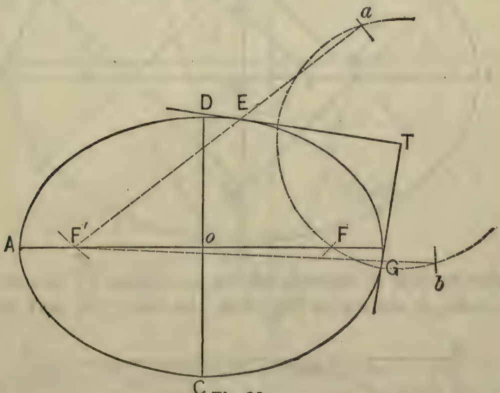


Fig. 83

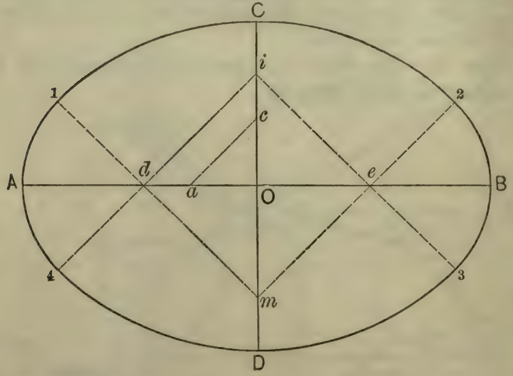
From the point T as a centre, and a radius equal to the distance to the nearer focus F, describe a circle. From F' as a centre, and a radius equal to the length of the longer axis, describe arcs cutting the circle just described at a and b. Draw lines from F' to a and b, cutting the circumference of the ellipse at E and G. Draw lines from T through E and G, and they will be the tangents required.

PROBLEM 34.—*To describe an ellipse approximately, by means of circular arcs.*

First (with arcs of two radii, Fig. 84).—Take half the difference of the two axes AB and CD, and set it off from the centre O to a and c on OA and OC; draw ac, and set off half ac to d; draw di parallel to ac; set off Oe equal to Od; join ei, and draw em and dm parallels to di and ie. On m as a centre, with a radius mC, describe an arc through C, terminating in 1 and 2; and with i as a centre, and D as a radius, describe an arc through D, terminating in points 3 and 4. On d and e as centres describe arcs

through A and B, connecting the points 1 and 4, 2 and 3. The four arcs thus described form approximately an ellipse. This method does not apply satisfactorily when the conjugate axis is less than two-thirds of the transverse axis.

Fig. 84



Another method of approximating an ellipse by means of arcs of two radii, is shown in Fig. 84*a*, the axis AB, and the semi-

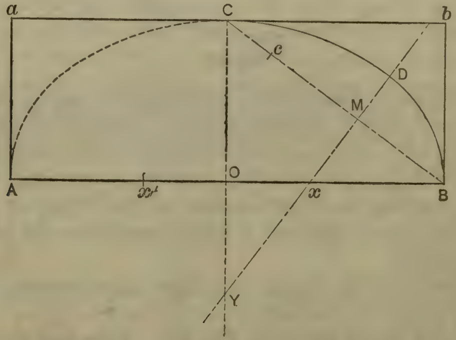


Fig. 84a.

minor axis OC being given. Draw the rectangle $AabB$, and the diagonal CB. Lay off Cc equal to the difference between OB and OC. Bisect cB at M, and erect the perpendicular YD, intersecting CO produced at Y and OB, at x. Make $Ox' = Ox$. Then will x, x', and Y be the three centres required, the curves becoming tangent at D. This method gives a slightly fuller curve at the haunches than the preceding one.

Second (with arcs of three radii, Fig. 85).—On the transverse axis AB draw the rectangle $AGEB$, equal in height to OC, half

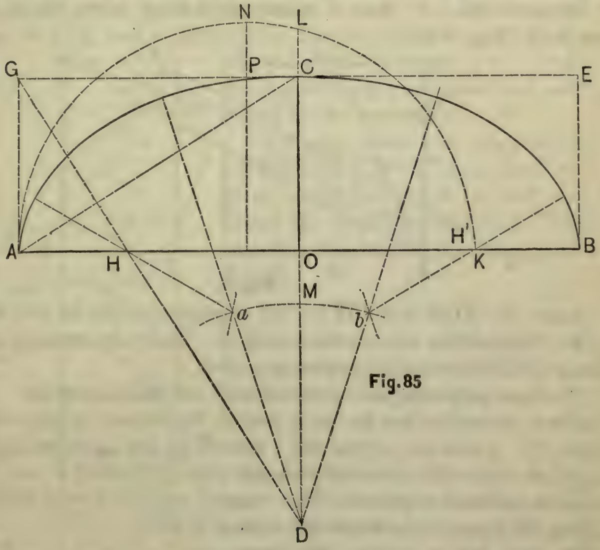


Fig. 85

the conjugate axis. Draw GD perpendicular to AC. Set off OK equal to OC, and on AK as a diameter describe the semicircle ANK. Draw a radius parallel to OC, intersecting the semicircle at N, and the line GE at P. Extend OC to L and to D. Set off OM equal to PN, and on D as a centre, with a radius DM, describe an arc. From A and B as centres, with a radius OL, intersect this arc at a and b. The points H, a, D, b, H', are the centres of the arcs required. Produce the lines aH, Da, Db, bH', and the spaces enclosed determine the lengths of each arc. This process works well for nearly all ellipses. It is employed in striking out vaults, stone arches, and bridges.

Note.—In this example the point H' happens to coincide with the point K, but this need not necessarily be the case.

The Parabola.

Problem 35.—*To construct a parabola when the vertex A, the axis AB, and a point, M, of the curve, are given* (Fig. 86).

Construct the rectangle $ABMC$. Divide MC into any number of equal parts, four for instance. Divide AC in like manner. Connect $A1$, $A2$, and $A3$. Through $1'$, $2'$, $3'$, draw parallels to

the axis. The intersections I, II, and III, of these lines, are points in the required curve.

PROBLEM 36.—*To draw a tangent to a given point,* II, *of the parabola* (Fig. 86).

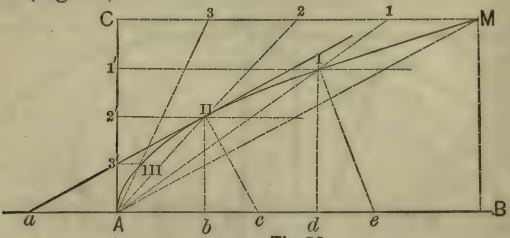


Fig. 86

From the given point II let fall a perpendicular on the axis at b. Extend the axis to the left of A. Make Aa equal to Ab. Draw aII, and it is the tangent required.

The lines perpendicular to the tangent are called normals. *To find the normal to any point* I, *having the tangent to any other point,* II. Draw the normal IIc. From I let fall a perpendicular Id, on the axis AB. Lay off de equal to bc. Connect Ie, an we have the normal required. The tangent may be drawn at I by laying off a perpendicular to the normal Ie at I.

The Hyperbola.

The hyperbola possesses the characteristic that if, from any point, P, two straight lines be drawn to two fixed points, F and F', the foci, their *difference* shall always be the same.

PROBLEM 37.—*To describe an hyperbola through a given vertex, a, with the given difference ab, and one of the foci, F* (Fig. 87).

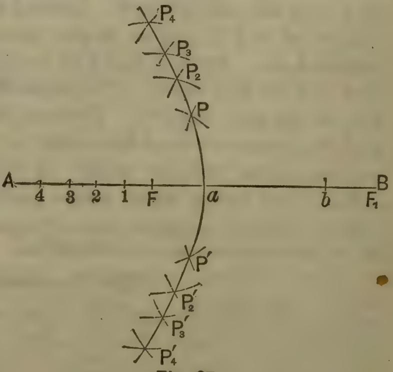


Fig. 87

Draw the axis of the hyperbola AB, with the given distance ab and the focus F marked on it. From b lay off bF_1 equal to aF for the other focus. Take any point, as 1 on AB, and with $a1$ as

a radius, and F as a centre, describe two short arcs above and below the axis. With $b1$ as a radius, and F' as a centre, describe arcs cutting those just described at P and P'. Take several points, as 2, 3, and 4, and obtain the corresponding points P_2, P_3, and P_4 in the same way. Join these points with a curved line, and it will be an hyperbola.

To draw a tangent to any point of an hyperbola, draw lines from the given point to each of the foci, and bisect the angle thus formed. The bisecting line will be the tangent required.

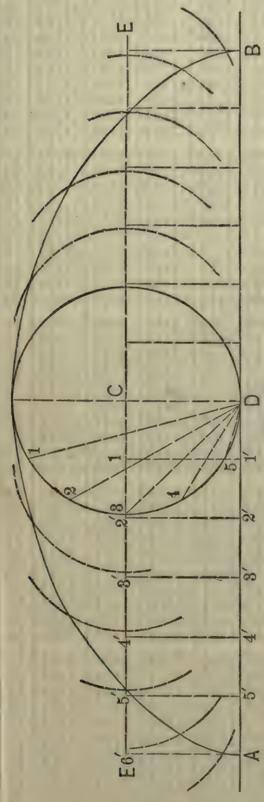


Fig. 88.

The Cycloid.

The *cycloid* is the curve described by a point in the circumference of a circle rolling in a straight line.

PROBLEM 38.—*To describe a cycloid* (Fig. 88).

Draw the straight line AB as the base. Describe the generating circle tangent to this line at the centre, and through the centre of the circle, C, draw the line EE parallel to the base. Let fall a perpendicular from C upon the base. Divide the semi-circumference into any number of equal parts, for instance, six. Lay off on AB and CE distances $C1'$, $1'2'$, etc., equal to the divisions of the circumference. Draw the chords $D1$, $D2$, etc. From the points $1'$, $2'$, $3'$, on the line CE, with radii equal to the generating circle, describe arcs. From the points $1'$, $2'$, $3'$, $4'$, $5'$, on the line BA, and with radii equal respectively to the chords $D1$, $D2$, $D3$, $D4$, $D5$, describe arcs cutting the preceding, and the intersections will be points of the curve required.

TABLE OF CHORDS; Radius = 1.0000.

M.	0°	1°	2°	3°	4°	5°	6°	7°	8°	9°	10°	M.
0′	.0000	.0175	.0349	.0524	.0698	.0872	.1047	.1221	.1395	.1569	.1743	0′
1	.0003	.0177	.0352	.0526	.0701	.0875	.1050	.1224	.1398	.1572	.1746	1
2	.0006	.0180	.0355	.0529	.0704	.0878	.1053	.1227	.1401	.1575	.1749	2
3	.0009	.0183	.0358	.0532	.0707	.0881	.1055	.1230	.1404	.1578	.1752	3
4	.0012	.0186	.0361	.0535	.0710	.0884	.1058	.1233	.1407	.1581	.1755	4
5	.0015	.0189	.0364	.0538	.0713	.0887	.1061	.1235	.1410	.1584	.1758	5
6	.0017	.0192	.0366	.0541	.0715	.0890	.1064	.1238	.1413	.1587	.1761	6
7	.0020	.0195	.0369	.0544	.0718	.0893	.1067	.1241	.1415	.1589	.1763	7
8	.0023	.0198	.0372	.0547	.0721	.0896	.1070	.1244	.1418	.1592	.1766	8
9	.0026	.0201	.0375	.0550	.0724	.0899	.1073	.1247	.1421	.1595	.1769	9
10	.0029	.0204	.0378	.0553	.0727	.0901	.1076	.1250	.1424	.1598	.1772	10
11	.0032	.0207	.0381	.0556	.0730	.0904	.1079	.1253	.1427	.1601	.1775	11
12	.0035	.0209	.0384	.0558	.0733	.0907	.1082	.1256	.1430	.1604	.1778	12
13	.0038	.0212	.0387	.0561	.0736	.0910	.1084	.1259	.1433	.1607	.1781	13
14	.0041	.0215	.0390	.0564	.0739	.0913	.1087	.1262	.1436	.1610	.1784	14
15	.0044	.0218	.0393	.0567	.0742	.0916	.1090	.1265	.1439	.1613	.1787	15
16	.0047	.0221	.0396	.0570	.0745	.0919	.1093	.1267	.1442	.1616	.1789	16
17	.0049	.0224	.0398	.0573	.0747	.0922	.1096	.1270	.1444	.1618	.1792	17
18	.0052	.0227	.0401	.0576	.0750	.0925	.1099	.1273	.1447	.1621	.1795	18
19	.0055	.0230	.0404	.0579	.0753	.0928	.1102	.1276	.1450	.1624	.1798	19
20	.0058	.0233	.0407	.0582	.0756	.0931	.1105	.1279	.1453	.1627	.1801	20
21	.0061	.0236	.0410	.0585	.0759	.0933	.1108	.1282	.1456	.1630	.1804	21
22	.0064	.0239	.0413	.0588	.0762	.0936	.1111	.1285	.1459	.1633	.1807	22
23	.0067	.0241	.0416	.0590	.0765	.0939	.1114	.1288	.1462	.1636	.1810	23
24	.0070	.0244	.0419	.0593	.0768	.0942	.1116	.1291	.1465	.1639	.1813	24
25	.0073	.0247	.0422	.0596	.0771	.0945	.1119	.1294	.1468	.1642	.1816	25
26	.0076	.0250	.0425	.0599	.0774	.0948	.1122	.1296	.1471	.1645	.1818	26
27	.0079	.0253	.0428	.0602	.0776	.0951	.1125	.1299	.1473	.1647	.1821	27
28	.0081	.0256	.0430	.0605	.0779	.0954	.1128	.1302	.1476	.1650	.1824	28
29	.0084	.0259	.0433	.0608	.0782	.0957	.1131	.1305	.1479	.1653	.1827	29
30	.0087	.0262	.0436	.0611	.0785	.0960	.1134	.1308	.1482	.1656	.1830	30
31	.0090	.0265	.0439	.0614	.0788	.0962	.1137	.1311	.1485	.1659	.1833	31
32	.0093	.0268	.0442	.0617	.0791	.0965	.1140	.1314	.1488	.1662	.1836	32
33	.0096	.0271	.0445	.0619	.0794	.0968	.1143	.1317	.1491	.1665	.1839	33
34	.0099	.0273	.0448	.0622	.0797	.0971	.1145	.1320	.1494	.1668	.1842	34
35	.0102	.0276	.0451	.0625	.0800	.0974	.1148	.1323	.1497	.1671	.1845	35
36	.0105	.0279	.0454	.0628	.0803	.0977	.1151	.1325	.1500	.1674	.1847	36
37	.0108	.0282	.0457	.0631	.0806	.0980	.1154	.1328	.1502	.1676	.1850	37
38	.0111	.0285	.0460	.0634	.0808	.0983	.1157	.1331	.1505	.1679	.1853	38
39	.0113	.0288	.0462	.0637	.0811	.0986	.1160	.1334	.1508	.1682	.1856	39
40	.0116	.0291	.0465	.0640	.0814	.0989	.1163	.1337	.1511	.1685	.1859	40
41	.0119	.0294	.0468	.0643	.0817	.0992	.1166	.1340	.1514	.1688	.1862	41
42	.0122	.0297	.0471	.0646	.0820	.0994	.1169	.1343	.1517	.1691	.1865	42
43	.0125	.0300	.0474	.0649	.0823	.0997	.1172	.1346	.1520	.1694	.1868	43
44	.0128	.0303	.0477	.0651	.0826	.1000	.1175	.1349	.1523	.1697	.1871	44
45	.0131	.0305	.0480	.0654	.0829	.1003	.1177	.1352	.1526	.1700	.1873	45
46	.0134	.0308	.0483	.0657	.0832	.1006	.1180	.1355	.1529	.1703	.1876	46
47	.0137	.0311	.0486	.0660	.0835	.1009	.1183	.1357	.1531	.1705	.1879	47
48	.0140	.0314	.0489	.0663	.0838	.1012	.1186	.1360	.1534	.1708	.1882	48
49	.0143	.0317	.0492	.0666	.0840	.1015	.1189	.1363	.1537	.1711	.1885	49
50	.0145	.0320	.0494	.0669	.0843	.1018	.1192	.1366	.1540	.1714	.1888	50
51	.0148	.0323	.0497	.0672	.0846	.1021	.1195	.1369	.1543	.1717	.1891	51
52	.0151	.0326	.0500	.0675	.0849	.1023	.1198	.1372	.1546	.1720	.1894	52
53	.0154	.0329	.0503	.0678	.0852	.1026	.1201	.1375	.1549	.1723	.1897	53
54	.0157	.0332	.0506	.0681	.0855	.1029	.1204	.1378	.1552	.1726	.1900	54
55	.0160	.0335	.0509	.0683	.0858	.1032	.1206	.1381	.1555	.1729	.1902	55
56	.0163	.0337	.0512	.0686	.0861	.1035	.1209	.1384	.1558	.1732	.1905	56
57	.0166	.0340	.0515	.0689	.0864	.1038	.1212	.1386	.1560	.1734	.1908	57
58	.0169	.0343	.0518	.0692	.0867	.1041	.1215	.1389	.1563	.1737	.1911	58
59	.0172	.0346	.0521	.0695	.0869	.1044	.1218	.1392	.1566	.1740	.1914	59
60	.0175	.0349	.0524	.0698	.0872	.1047	.1221	.1395	.1569	.1743	.1917	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	11°	12°	13°	14°	15°	16°	17°	18°	19°	20°	21°	M.
0	.1917	.2091	.2264	.2437	.2611	.2783	.2956	.3129	.3301	.3473	.3645	0′
1	.1920	.2093	.2267	.2440	.2613	.2786	.2959	.3132	.3304	.3476	.3648	1
2	.1923	.2096	.2270	.2443	.2616	.2789	.2962	.3134	.3307	.3479	.3650	2
3	.1926	.2099	.2273	.2446	.2619	.2792	.2965	.3137	.3310	.3482	.3653	3
4	.1928	.2102	.2276	.2449	.2622	.2795	.2968	.3140	.3312	.3484	.3656	4
5	.1931	.2105	.2279	.2452	.2625	.2798	.2971	.3143	.3315	.3487	.3659	5
6	.1934	.2108	.2281	.2455	.2628	.2801	.2973	.3146	.3318	.3490	.3662	6
7	.1937	.2111	.2284	.2458	.2631	.2804	.2976	.3149	.3321	.3493	.3665	7
8	.1940	.2114	.2287	.2460	.2634	.2807	.2979	.3152	.3324	.3496	.3668	8
9	.1943	.2117	.2290	.2463	.2636	.2809	.2982	.3155	.3327	.3499	.3670	9
10	.1946	.2119	.2293	.2466	.2639	.2812	.2985	.3157	.3330	.3502	.3673	10
11	.1949	.2122	.2296	.2469	.2642	.2815	.2988	.3160	.3333	.3504	.3676	11
12	.1952	.2125	.2299	.2472	.2645	.2818	.2991	.3163	.3335	.3507	.3679	12
13	.1955	.2128	.2302	.2475	.2648	.2821	.2994	.3166	.3338	.3510	.3682	13
14	.1957	.2131	.2305	.2478	.2651	.2824	.2996	.3169	.3341	.3513	.3685	14
15	.1960	.2134	.2307	.2481	.2654	.2827	.2999	.3172	.3344	.3516	.3688	15
16	.1963	.2137	.2310	.2484	.2657	.2830	.3002	.3175	.3347	.3519	.3690	16
17	.1966	.2140	.2313	.2486	.2660	.2832	.3005	.3178	.3350	.3522	.3693	17
18	.1969	.2143	.2316	.2489	.2662	.2835	.3008	.3180	.3353	.3525	.3696	18
19	.1972	.2146	.2319	.2492	.2665	.2838	.3011	.3183	.3355	.3527	.3699	19
20	.1975	.2148	.2322	.2495	.2668	.2841	.3014	.3186	.3358	.3530	.3702	20
21	.1978	.2151	.2325	.2498	.2671	.2844	.3017	.3189	.3361	.3533	.3705	21
22	.1981	.2154	.2328	.2501	.2674	.2847	.3019	.3192	.3364	.3536	.3708	22
23	.1983	.2157	.2331	.2504	.2677	.2850	.3022	.3195	.3367	.3539	.3710	23
24	.1986	.2160	.2333	.2507	.2680	.2853	.3025	.3198	.3370	.3542	.3713	24
25	.1989	.2163	.2336	.2510	.2683	.2855	.3028	.3200	.3373	.3545	.3716	25
26	.1992	.2166	.2339	.2512	.2685	.2858	.3031	.3203	.3376	.3547	.3719	26
27	.1995	.2169	.2342	.2515	.2688	.2861	.3034	.3206	.3378	.3550	.3722	27
28	.1998	.2172	.2345	.2518	.2691	.2864	.3037	.3209	.3381	.3553	.3725	28
29	.2001	.2174	.2348	.2521	.2694	.2867	.3040	.3212	.3384	.3556	.3728	29
30	.2004	.2177	.2351	.2524	.2697	.2870	.3042	.3215	.3387	.3559	.3730	30
31	.2007	.2180	.2354	.2527	.2700	.2873	.3045	.3218	.3390	.3562	.3733	31
32	.2010	.2183	.2357	.2530	.2703	.2876	.3048	.3221	.3393	.3565	.3736	32
33	.2012	.2186	.2359	.2533	.2706	.2878	.3051	.3223	.3396	.3567	.3739	33
34	.2015	.2189	.2362	.2536	.2709	.2881	.3054	.3226	.3398	.3570	.3742	34
35	.2018	.2192	.2365	.2538	.2711	.2884	.3057	.3229	.3401	.3573	.3745	35
36	.2021	.2195	.2368	.2541	.2714	.2887	.3060	.3232	.3404	.3576	.3748	36
37	.2024	.2198	.2371	.2544	.2717	.2890	.3063	.3235	.3407	.3579	.3750	37
38	.2027	.2200	.2374	.2547	.2720	.2893	.3065	.3238	.3410	.3582	.3753	38
39	.2030	.2203	.2377	.2550	.2723	.2896	.3068	.3241	.3413	.3585	.3756	39
40	.2033	.2206	.2380	.2553	.2726	.2899	.3071	.3244	.3416	.3587	.3759	40
41	.2036	.2209	.2383	.2556	.2729	.2902	.3074	.3246	.3419	.3590	.3762	41
42	.2038	.2212	.2385	.2559	.2732	.2904	.3077	.3249	.3421	.3593	.3765	42
43	.2041	.2215	.2388	.2561	.2734	.2907	.3080	.3252	.3424	.3596	.3768	43
44	.2044	.2218	.2391	.2564	.2737	.2910	.3083	.3255	.3427	.3599	.3770	44
45	.2047	.2221	.2394	.2567	.2740	.2913	.3086	.3258	.3430	.3602	.3773	45
46	.2050	.2224	.2397	.2570	.2743	.2916	.3088	.3261	.3433	.3605	.3776	46
47	.2053	.2226	.2400	.2573	.2746	.2919	.3091	.3264	.3436	.3608	.3779	47
48	.2056	.2229	.2403	.2576	.2749	.2922	.3094	.3267	.3439	.3610	.3782	48
49	.2059	.2232	.2406	.2579	.2752	.2925	.3097	.3269	.3441	.3613	.3785	49
50	.2062	.2235	.2409	.2582	.2755	.2927	.3100	.3272	.3444	.3616	.3788	50
51	.2065	.2238	.2411	.2585	.2758	.2930	.3103	.3275	.3447	.3619	.3790	51
52	.2067	.2241	.2414	.2587	.2760	.2933	.3106	.3278	.3450	.3622	.3793	52
53	.2070	.2244	.2417	.2590	.2763	.2936	.3109	.3281	.3453	.3625	.3796	53
54	.2073	.2247	.2420	.2593	.2766	.2939	.3111	.3284	.3456	.3628	.3799	54
55	.2076	.2250	.2423	.2596	.2769	.2942	.3114	.3287	.3459	.3630	.3802	55
56	.2079	.2253	.2426	.2599	.2772	.2945	.3117	.3289	.3462	.3633	.3805	56
57	.2082	.2255	.2429	.2602	.2775	.2948	.3120	.3292	.3464	.3636	.3808	57
58	.2085	.2258	.2432	.2605	.2778	.2950	.3123	.3295	.3467	.3639	.3810	58
59	.2088	.2261	.2434	.2608	.2781	.2953	.3126	.3298	.3470	.3642	.3813	59
60	.2091	.2264	.2437	.2611	.2783	.2956	.3129	.3301	.3473	.3645	.3816	60

Table of Chords; Radius = 1.0000 (*continued*)

M.	22°	23°	24°	25°	26°	27°	28°	29°	30°	31°	32°	M.
0′	.3816	.3987	.4158	.4329	.4499	.4669	.4838	.5008	.5176	.5345	.5513	0′
1	.3819	.3990	.4161	.4332	.4502	.4672	.4841	.5010	.5179	.5348	.5516	1
2	.3822	.3993	.4164	.4334	.4505	.4675	.4844	.5013	.5182	.5350	.5518	2
3	.3825	.3996	.4167	.4337	.4508	.4677	.4847	.5016	.5185	.5353	.5521	3
4	.3828	.3999	.4170	.4340	.4510	.4680	.4850	.5019	.5188	.5356	.5524	4
5	.3830	.4002	.4172	.4343	.4513	.4683	.4853	.5022	.5190	.5359	.5527	5
6	.3833	.4004	.4175	.4346	.4516	.4686	.4855	.5024	.5193	.5362	.5530	6
7	.3836	.4007	.4178	.4349	.4519	.4689	.4858	.5027	.5196	.5364	.5532	7
8	.3839	.4010	.4181	.4352	.4522	.4692	.4861	.5030	.5199	.5367	.5535	8
9	.3842	.4013	.4184	.4354	.4525	.4694	.4864	.5033	.5202	.5370	.5538	9
10	.3845	.4016	.4187	.4357	.4527	.4697	.4867	.5036	.5204	.5373	.5541	10
11	.3848	.4019	.4190	.4360	.4530	.4700	.4869	.5039	.5207	.5376	.5543	11
12	.3850	.4022	.4192	.4363	.4533	.4703	.4872	.5041	.5210	.5378	.5546	12
13	.3853	.4024	.4195	.4366	.4536	.4706	.4875	.5044	.5213	.5381	.5549	13
14	.3856	.4027	.4198	.4369	.4539	.4708	.4878	.5047	.5216	.5384	.5552	14
15	.3859	.4030	.4201	.4371	.4542	.4711	.4881	.5050	.5219	.5387	.5555	15
16	.3862	.4033	.4204	.4374	.4544	.4714	.4884	.5053	.5221	.5390	.5557	16
17	.3865	.4036	.4207	.4377	.4547	.4717	.4886	.5055	.5224	.5392	.5560	17
18	.3868	.4039	.4209	.4380	.4550	.4720	.4889	.5058	.5227	.5395	.5563	18
19	.3870	.4042	.4212	.4383	.4553	.4723	.4892	.5061	.5230	.5398	.5566	19
20	.3873	.4044	.4215	.4386	.4556	.4725	.4895	.5064	.5233	.5401	.5569	20
21	.3876	.4047	.4218	.4388	.4559	.4728	.4898	.5067	.5235	.5404	.5571	21
22	.3879	.4050	.4221	.4391	.4561	.4731	.4901	.5070	.5238	.5406	.5574	22
23	.3882	.4053	.4224	.4394	.4564	.4734	.4903	.5072	.5241	.5409	.5577	23
24	.3885	.4056	.4226	.4397	.4567	.4737	.4906	.5075	.5244	.5412	.5580	24
25	.3888	.4059	.4229	.4400	.4570	.4740	.4909	.5078	.5247	.5415	.5583	25
26	.3890	.4061	.4232	.4403	.4573	.4742	.4912	.5081	.5249	.5418	.5585	26
27	.3893	.4064	.4235	.4405	.4576	.4745	.4915	.5084	.5252	.5420	.5588	27
28	.3896	.4067	.4238	.4408	.4578	.4748	.4917	.5086	.5255	.5423	.5591	28
29	.3899	.4070	.4241	.4411	.4581	.4751	.4920	.5089	.5258	.5426	.5594	29
30	.3902	.4073	.4244	.4414	.4584	.4754	.4923	.5092	.5261	.5429	.5597	30
31	.3905	.4076	.4246	.4417	.4587	.4757	.4926	.5095	.5263	.5432	.5599	31
32	.3908	.4079	.4249	.4420	.4590	.4759	.4929	.5098	.5266	.5434	.5602	32
33	.3910	.4081	.4252	.4422	.4593	.4762	.4932	.5100	.5269	.5437	.5605	33
34	.3913	.4084	.4255	.4425	.4595	.4765	.4934	.5103	.5272	.5440	.5608	34
35	.3916	.4087	.4258	.4428	.4598	.4768	.4937	.5106	.5275	.5443	.5611	35
36	.3919	.4090	.4261	.4431	.4601	.4771	.4940	.5109	.5277	.5446	.5613	36
37	.3922	.4093	.4263	.4434	.4604	.4773	.4943	.5112	.5280	.5448	.5616	37
38	.3925	.4096	.4266	.4437	.4607	.4776	.4946	.5115	.5283	.5451	.5619	38
39	.3927	.4098	.4269	.4439	.4609	.4779	.4948	.5117	.5286	.5454	.5622	39
40	.3930	.4101	.4272	.4442	.4612	.4782	.4951	.5120	.5289	.5457	.5625	40
41	.3933	.4104	.4275	.4445	.4615	.4785	.4954	.5123	.5291	.5460	.5627	41
42	.3936	.4107	.4278	.4448	.4618	.4788	.4957	.5126	.5294	.5462	.5630	42
43	.3939	.4110	.4280	.4451	.4621	.4790	.4960	.5129	.5297	.5465	.5633	43
44	.3942	.4113	.4283	.4454	.4624	.4793	.4963	.5131	.5300	.5468	.5636	44
45	.3945	.4116	.4286	.4456	.4626	.4796	.4965	.5134	.5303	.5471	.5638	45
46	.3947	.4118	.4289	.4459	.4629	.4799	.4968	.5137	.5306	.5474	.5641	46
47	.3950	.4121	.4292	.4462	.4632	.4802	.4971	.5140	.5308	.5476	.5644	47
48	.3953	.4124	.4295	.4465	.4635	.4805	.4974	.5143	.5311	.5479	.5647	48
49	.3956	.4127	.4298	.4468	.4638	.4807	.4977	.5145	.5314	.5482	.5650	49
50	.3959	.4130	.4300	.4471	.4641	.4810	.4979	.5148	.5317	.5485	.5652	50
51	.3962	.4133	.4303	.4474	.4643	.4813	.4982	.5151	.5320	.5488	.5655	51
52	.3965	.4135	.4306	.4476	.4646	.4816	.4985	.5154	.5322	.5490	.5658	52
53	.3967	.4138	.4309	.4479	.4649	.4819	.4988	.5157	.5325	.5493	.5661	53
54	.3970	.4141	.4312	.4482	.4652	.4822	.4991	.5160	.5328	.5496	.5664	54
55	.3973	.4144	.4315	.4485	.4655	.4824	.4994	.5162	.5331	.5499	.5666	55
56	.3976	.4147	.4317	.4488	.4658	.4827	.4996	.5165	.5334	.5502	.5669	56
57	.3979	.4150	.4320	.4491	.4660	.4830	.4999	.5168	.5336	.5504	.5672	57
58	.3982	.4153	.4323	.4493	.4663	.4833	.5002	.5171	.5339	.5507	.5675	58
59	.3985	.4155	.4326	.4496	.4666	.4836	.5005	.5174	.5342	.5510	.5678	59
60	.3987	.4158	.4329	.4499	.4669	.4838	.5008	.5176	.5345	.5513	.5680	60

Table of Chords; Radius=1.0000 (*continued*).

M.	33°	34°	35°	36°	37°	38°	39°	40°	41°	42°	43°	M.
0′	.5680	.5847	.6014	.6180	.6346	.6511	.6676	.6840	.7004	.7167	.7330	0′
1	.5683	.5850	.6017	.6183	.6349	.6514	.6679	.6843	.7007	.7170	.7333	1
2	.5686	.5853	.6020	.6186	.6352	.6517	.6682	.6846	.7010	.7173	.7335	2
3	.5689	.5856	.6022	.6189	.6354	.6520	.6684	.6849	.7012	.7176	.7338	3
4	.5691	.5859	.6025	.6191	.6357	.6522	.6687	.6851	.7015	.7178	.7341	4
5	.5694	.5861	.6028	.6194	.6360	.6525	.6690	.6854	.7018	.7181	.7344	5
6	.5697	.5864	.6031	.6197	.6363	.6528	.6693	.6857	.7020	.7184	.7346	6
7	.5700	.5867	.6034	.6200	.6365	.6531	.6695	.6860	.7023	.7186	.7349	7
8	.5703	.5870	.6036	.6202	.6368	.6533	.6698	.6862	.7026	.7189	.7352	8
9	.5705	.5872	.6039	.6205	.6371	.6536	.6701	.6865	.7029	.7192	.7354	9
10	.5708	.5875	.6042	.6208	.6374	.6539	.6704	.6868	.7031	.7195	.7357	10
11	.5711	.5878	.6045	.6211	.6376	.6542	.6706	.6870	.7034	.7197	.7360	11
12	.5714	.5881	.6047	.6214	.6379	.6544	.6709	.6873	.7037	.7200	.7362	12
13	.5717	.5884	.6050	.6216	.6382	.6547	.6712	.6876	.7040	.7203	.7365	13
14	.5719	.5886	.6053	.6219	.6385	.6550	.6715	.6879	.7042	.7205	.7368	14
15	.5722	.5889	.6056	.6222	.6387	.6553	.6717	.6881	.7045	.7208	.7371	15
16	.5725	.5892	.6058	.6225	.6390	.6555	.6720	.6884	.7048	.7211	.7373	16
17	.5728	.5895	.6061	.6227	.6393	.6558	.6723	.6887	.7050	.7214	.7376	17
18	.5730	.5897	.6064	.6230	.6396	.6561	.6725	.6890	.7053	.7216	.7379	18
19	.5733	.5900	.6067	.6233	.6398	.6564	.6728	.6892	.7056	.7219	.7381	19
20	.5736	.5903	.6070	.6236	.6401	.6566	.6731	.6895	.7059	.7222	.7384	20
21	.5739	.5906	.6072	.6238	.6404	.6569	.6734	.6898	.7061	.7224	.7387	21
22	.5742	.5909	.6075	.6241	.6407	.6572	.6736	.6901	.7064	.7227	.7390	22
23	.5744	.5911	.6078	.6244	.6410	.6575	.6739	.6903	.7067	.7230	.7392	23
24	.5747	.5914	.6081	.6247	.6412	.6577	.6742	.6906	.7069	.7232	.7395	24
25	.5750	.5917	.6083	.6249	.6415	.6580	.6745	.6909	.7072	.7235	.7398	25
26	.5753	.5920	.6086	.6252	.6418	.6583	.6747	.6911	.7075	.7238	.7400	26
27	.5756	.5922	.6089	.6255	.6421	.6586	.6750	.6914	.7078	.7241	.7403	27
28	.5758	.5925	.6092	.6258	.6423	.6588	.6753	.6917	.7080	.7243	.7406	28
29	.5761	.5928	.6095	.6260	.6426	.6591	.6756	.6920	.7083	.7246	.7408	29
30	.5764	.5931	.6097	.6263	.6429	.6594	.6758	.6922	.7086	.7249	.7411	30
31	.5767	.5934	.6100	.6266	.6432	.6597	.6761	.6925	.7089	.7251	.7414	31
32	.5769	.5936	.6103	.6269	.6434	.6599	.6764	.6928	.7091	.7254	.7417	32
33	.5772	.5939	.6106	.6272	.6437	.6602	.6767	.6931	.7094	.7257	.7419	33
34	.5775	.5942	.6108	.6274	.6440	.6605	.6769	.6933	.7097	.7260	.7422	34
35	.5778	.5945	.6111	.6277	.6443	.6608	.6772	.6936	.7099	.7262	.7425	35
36	.5781	.5947	.6114	.6280	.6445	.6610	.6775	.6939	.7102	.7265	.7427	36
37	.5783	.5950	.6117	.6283	.6448	.6613	.6777	.6941	.7105	.7268	.7430	37
38	.5786	.5953	.6119	.6285	.6451	.6616	.6780	.6944	.7108	.7270	.7433	38
39	.5789	.5956	.6122	.6288	.6454	.6619	.6783	.6947	.7110	.7273	.7435	39
40	.5792	.5959	.6125	.6291	.6456	.6621	.6786	.6950	.7113	.7276	.7438	40
41	.5795	.5961	.6128	.6294	.6459	.6624	.6788	.6952	.7116	.7279	.7441	41
42	.5797	.5964	.6130	.6296	.6462	.6627	.6791	.6955	.7118	.7281	.7443	42
43	.5800	.5967	.6133	.6299	.6465	.6630	.6794	.6958	.7121	.7284	.7446	43
44	.5803	.5970	.6136	.6302	.6467	.6632	.6797	.6961	.7124	.7287	.7449	44
45	.5806	.5972	.6139	.6205	.6470	.6635	.6799	.6963	.7127	.7289	.7452	45
46	.5808	.5975	.6142	.6307	.6473	.6638	.6802	.6966	.7129	.7292	.7454	46
47	.5811	.5978	.6144	.6310	.6476	.6640	.6805	.6969	.7132	.7295	.7457	47
48	.5814	.5981	.6147	.6313	.6478	.6643	.6808	.6971	.7135	.7298	.7460	48
49	.5817	.5984	.6150	.6316	.6481	.6646	.6810	.6974	.7137	.7300	.7462	49
50	.5820	.5986	.6153	.6318	.6484	.6649	.6813	.6977	.7140	.7303	.7465	50
51	.5822	.5989	.6155	.6321	.6487	.6651	.6816	.6980	.7143	.7306	.7468	51
52	.5825	.5992	.6158	.6324	.6489	.6654	.6819	.6982	.7146	.7308	.7471	52
53	.5828	.5995	.6161	.6327	.6492	.6657	.6821	.6985	.7148	.7311	.7473	53
54	.5831	.5997	.6164	.6330	.6495	.6660	.6824	.6988	.7151	.7314	.7476	51
55	.5834	.6000	.6166	.6332	.6498	.6662	.6827	.6991	.7154	.7316	.7479	55
56	.5836	.6003	.6169	.6335	.6500	.6665	.6829	.6993	.7156	.7319	.7481	56
57	.5839	.6006	.6172	.6338	.6503	.6668	.6832	.6996	.7159	.7322	.7484	57
58	.5842	.6009	.6175	.6341	.6506	.6671	.6835	.6999	.7162	.7325	.7487	58
59	.5845	.6011	.6178	.6343	.6509	.6673	.6838	.7001	.7165	.7327	.7489	59
60	.5847	.6014	.6180	.6346	.6511	.6676	.6840	.7004	.7167	.7330	.7492	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	44°	45°	46°	47°	48°	49°	50°	51°	52°	53°	54°	M.
0′	.7492	.7654	.7815	.7975	.8135	.8294	.8452	.8610	.8767	.8924	.9080	0
1	.7495	.7656	.7817	.7978	.8137	.8297	.8455	.8613	.8770	.8927	.9082	1
2	.7498	.7659	.7820	.7980	.8140	.8299	.8458	.8615	.8773	.8929	.9085	2
3	.7500	.7662	.7823	.7983	.8143	.8302	.8460	.8618	.8775	.8932	.9088	3
4	.7503	.7664	.7825	.7986	.8145	.8304	.8463	.8621	.8778	.8934	.9090	4
5	.7506	.7667	.7828	.7988	.8148	.8307	.8466	.8623	.8780	.8937	.9093	5
6	.7508	.7670	.7831	.7991	.8151	.8310	.8468	.8626	.8783	.8940	.9095	6
7	.7511	.7672	.7833	.7994	.8153	.8312	.8471	.8629	.8786	.8942	.9098	7
8	.7514	.7675	.7836	.7996	.8156	.8315	.8473	.8631	.8788	.8945	.9101	8
9	.7516	.7678	.7839	.7999	.8159	.8318	.8476	.8634	.8791	.8947	.9103	9
10	.7519	.7681	.7841	.8002	.8161	.8320	.8479	.8636	.8794	.8950	.9106	10
11	.7522	.7683	.7844	.8004	.8164	.8323	.8481	.8639	.8796	.8953	.9108	11
12	.7524	.7686	.7847	.8007	.8167	.8326	.8484	.8642	.8799	.8955	.9111	12
13	.7527	.7689	.7849	.8010	.8169	.8328	.8487	.8644	.8801	.8958	.9113	13
14	.7530	.7691	.7852	.8012	.8172	.8331	.8489	.8647	.8804	.8960	.9116	14
15	.7533	.7694	.7855	.8015	.8175	.8334	.8492	.8650	.8807	.8963	.9119	15
16	.7535	.7697	.7857	.8018	.8177	.8336	.8495	.8652	.8809	.8966	.9121	16
17	.7538	.7699	.7860	.8020	.8180	.8339	.8497	.8655	.8812	.8968	.9124	17
18	.7541	.7702	.7863	.8023	.8183	.8341	.8500	.8657	.8814	.8971	.9126	18
19	.7543	.7705	.7865	.8026	.8185	.8344	.8502	.8660	.8817	.8973	.9129	19
20	.7546	.7707	.7868	.8028	.8188	.8347	.8505	.8663	.8820	.8976	.9132	20
21	.7549	.7710	.7871	.8031	.8190	.8349	.8508	.8665	.8822	.8979	.9134	21
22	.7551	.7713	.7873	.8034	.8193	.8352	.8510	.8668	.8825	.8981	.9137	22
23	.7554	.7715	.7876	.8036	.8196	.8355	.8513	.8671	.8828	.8984	.9139	23
24	.7557	.7718	.7879	.8039	.8198	.8357	.8516	.8673	.8830	.8986	.9142	24
25	.7560	.7721	.7882	.8042	.8201	.8360	.8518	.8676	.8833	.8989	.9145	25
26	.7562	.7723	.7884	.8044	.8204	.8363	.8521	.8678	.8835	.8992	.9147	26
27	.7565	.7726	.7887	.8047	.8206	.8365	.8523	.8681	.8838	.8994	.9150	27
28	.7568	.7729	.7890	.8050	.8209	.8368	.8526	.8684	.8841	.8997	.9152	28
29	.7570	.7731	.7892	.8052	.8212	.8371	.8529	.8686	.8843	.8999	.9155	29
30	.7573	.7734	.7895	.8055	.8214	.8373	.8531	.8689	.8846	.9002	.9157	30
31	.7576	.7737	.7898	.8058	.8217	.8376	.8534	.8692	.8848	.9005	.9160	31
32	.7578	.7740	.7900	.8060	.8220	.8378	.8537	.8694	.8851	.9007	.9163	32
33	.7581	.7742	.7903	.8063	.8222	.8381	.8539	.8697	.8854	.9010	.9165	33
34	.7584	.7745	.7906	.8066	.8225	.8384	.8542	.8699	.8856	.9012	.9168	34
35	.7586	.7748	.7908	.8068	.8228	.8386	.8545	.8702	.8859	.9015	.9170	35
36	.7589	.7750	.7911	.8071	.8230	.8389	.8547	.8705	.8861	.9018	.9173	36
37	.7592	.7753	.7914	.8074	.8233	.8392	.8550	.8707	.8864	.9020	.9176	37
38	.7595	.7756	.7916	.8076	.8236	.8394	.8552	.8710	.8867	.9023	.9178	38
39	.7597	.7758	.7919	.8079	.8238	.8397	.8555	.8712	.8869	.9025	.9181	39
40	.7600	.7761	.7922	.8082	.8241	.8400	.8558	.8715	.8872	.9028	.9183	40
41	.7603	.7764	.7924	.8084	.8244	.8402	.8560	.8718	.8874	.9031	.9186	41
42	.7605	.7766	.7927	.8087	.8246	.8405	.8563	.8720	.8877	.9033	.9188	42
43	.7608	.7769	.7930	.8090	.8249	.8408	.8566	.8723	.8880	.9036	.9191	43
44	.7611	.7772	.7932	.8092	.8251	.8410	.8568	.8726	.8882	.9038	.9194	44
45	.7613	.7774	.7935	.8095	.8254	.8413	.8571	.8728	.8885	.9041	.9196	45
46	.7616	.7777	.7938	.8098	.8257	.8415	.8573	.8731	.8887	.9044	.9199	46
47	.7619	.7780	.7940	.8100	.8259	.8418	.8576	.8734	.8890	.9046	.9201	47
48	.7621	.7782	.7943	.8103	.8262	.8421	.8579	.8736	.8893	.9049	.9204	48
49	.7624	.7785	.7946	.8105	.8265	.8423	.8581	.8739	.8895	.9051	.9207	49
50	.7627	.7788	.7948	.8108	.8267	.8426	.8584	.8741	.8898	.9054	.9209	50
51	.7629	.7791	.7951	.8111	.8270	.8429	.8587	.8744	.8900	.9056	.9212	51
52	.7632	.7793	.7954	.8113	.8273	.8431	.8589	.8747	.8903	.9059	.9214	52
53	.7635	.7796	.7956	.8116	.8275	.8434	.8592	.8749	.8906	.9062	.9217	53
54	.7638	.7799	.7959	.8119	.8278	.8437	.8594	.8752	.8908	.9064	.9219	54
55	.7640	.7801	.7962	.8121	.8281	.8439	.8597	.8754	.8911	.9067	.9222	55
56	.7643	.7804	.7964	.8124	.8283	.8442	.8600	.8757	.8914	.9069	.9225	56
57	.7646	.7807	.7967	.8127	.8286	.8444	.8602	.8760	.8916	.9072	.9227	57
58	.7648	.7809	.7970	.8129	.8289	.8447	.8605	.8762	.8919	.9075	.9230	58
59	.7651	.7812	.7972	.8132	.8291	.8450	.8608	.8765	.8921	.9077	.9232	59
60	.7654	.7815	.7975	.8135	.8294	.8452	.8610	.8767	.8924	.9080	.9235	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	55°	56°	57°	58°	59°	60°	61°	62°	63°	64°	M.
0′	.9235	.9389	.9543	.9696	.9848	1.0000	1.0151	1.0301	1.0450	1.0598	0′
1	.9238	.9392	.9546	.9699	.9851	1.0003	1.0153	1.0303	1.0452	1.0601	1
2	.9240	.9395	.9548	.9701	.9854	1.0005	1.0156	1.0306	1.0455	1.0603	2
3	.9243	.9397	.9551	.9704	.9856	1.0008	1.0158	1.0308	1.0457	1.0606	3
4	.9245	.9400	.9553	.9706	.9859	1.0010	1.0161	1.0311	1.0460	1.0608	4
5	.9248	.9402	.9556	.9709	.9861	1.0013	1.0163	1.0313	1.0462	1.0611	5
6	.9250	.9405	.9559	.9711	.9864	1.0015	1.0166	1.0316	1.0465	1.0613	6
7	.9253	.9407	.9561	.9714	.9866	1.0018	1.0168	1.0318	1.0467	1.0616	7
8	.9256	.9410	.9564	.9717	.9869	1.0020	1.0171	1.0321	1.0470	1.0618	8
9	.9258	.9413	.9566	.9719	.9871	1.0023	1.0173	1.0323	1.0472	1.0621	9
10	.9261	.9415	.9569	.9722	.9874	1.0025	1.0176	1.0326	1.0475	1.0623	10
11	.9263	.9418	.9571	.9724	.9876	1.0028	1.0178	1.0328	1.0477	1.0626	11
12	.9266	.9420	.9574	.9727	.9879	1.0030	1.0181	1.0331	1.0480	1.0628	12
13	.9268	.9423	.9576	.9729	.9881	1.0033	1.0183	1.0333	1.0482	1.0630	13
14	.9271	.9425	.9579	.9732	.9884	1.0035	1.0186	1.0336	1.0485	1.0633	14
15	.9274	.9428	.9581	.9734	.9886	1.0038	1.0188	1.0338	1.0487	1.0635	15
16	.9276	.9430	.9584	.9737	.9889	1.0040	1.0191	1.0341	1.0490	1.0638	16
17	.9279	.9433	.9587	.9739	.9891	1.0043	1.0193	1.0343	1.0492	1.0640	17
18	.9281	.9436	.9589	.9742	.9894	1.0045	1.0196	1.0346	1.0495	1.0643	18
19	.9284	.9438	.9592	.9744	.9897	1.0048	1.0198	1.0348	1.0497	1.0645	19
20	.9287	.9441	.9594	.9747	.9899	1.0050	1.0201	1.0351	1.0500	1.0648	20
21	.9289	.9443	.9597	.9750	.9902	1.0053	1.0203	1.0353	1.0502	1.0650	21
22	.9292	.9446	.9599	.9752	.9904	1.0055	1.0206	1.0356	1.0504	1.0653	22
23	.9294	.9448	.9602	.9755	.9907	1.0058	1.0208	1.0358	1.0507	1.0655	23
24	.9297	.9451	.9604	.9757	.9909	1.0060	1.0211	1.0361	1.0509	1.0658	24
25	.9299	.9454	.9607	.9760	.9912	1.0063	1.0213	1.0363	1.0512	1.0660	25
26	.9302	.9456	.9610	.9762	.9914	1.0065	1.0216	1.0366	1.0514	1.0662	26
27	.9305	.9459	.9612	.9765	.9917	1.0068	1.0218	1.0368	1.0517	1.0665	27
28	.9307	.9461	.9615	.9767	.9919	1.0070	1.0221	1.0370	1.0519	1.0667	28
29	.9310	.9464	.9617	.9770	.9922	1.0073	1.0223	1.0373	1.0522	1.0670	29
30	.9312	.9466	.9620	.9772	.9924	1.0075	1.0226	1.0375	1.0524	1.0672	30
31	.9315	.9469	.9622	.9775	.9927	1.0078	1.0228	1.0378	1.0527	1.0675	31
32	.9317	.9472	.9625	.9778	.9929	1.0080	1.0231	1.0380	1.0529	1.0677	32
33	.9320	.9474	.9627	.9780	.9932	1.0083	1.0233	1.0383	1.0532	1.0680	33
34	.9323	.9477	.9630	.9783	.9934	1.0086	1.0236	1.0385	1.0534	1.0682	34
35	.9325	.9479	.9633	.9785	.9937	1.0088	1.0238	1.0388	1.0537	1.0685	35
36	.9328	.9482	.9635	.9788	.9939	1.0091	1.0241	1.0390	1.0539	1.0687	36
37	.9330	.9484	.9638	.9790	.9942	1.0093	1.0243	1.0393	1.0542	1.0690	37
38	.9333	.9487	.9640	.9793	.9945	1.0096	1.0246	1.0395	1.0544	1.0692	38
39	.9335	.9489	.9643	.9795	.9947	1.0098	1.0248	1.0398	1.0547	1.0694	39
40	.9338	.9492	.9645	.9798	.9950	1.0101	1.0251	1.0400	1.0549	1.0697	40
41	.9341	.9495	.9648	.9800	.9952	1.0103	1.0253	1.0403	1.0551	1.0699	41
42	.9343	.9497	.9650	.9803	.9955	1.0106	1.0256	1.0405	1.0554	1.0702	42
43	.9346	.9500	.9653	.9805	.9957	1.0108	1.0258	1.0408	1.0556	1.0704	43
44	.9348	.9502	.9655	.9808	.9960	1.0111	1.0261	1.0410	1.0559	1.0707	44
45	.9351	.9505	.9658	.9810	.9962	1.0113	1.0263	1.0413	1.0561	1.0709	45
46	.9353	.9507	.9661	.9813	.9965	1.0116	1.0266	1.0415	1.0564	1.0712	46
47	.9356	.9510	.9663	.9816	.9967	1.0118	1.0268	1.0418	1.0566	1.0714	47
48	.9359	.9512	.9666	.9818	.9970	1.0121	1.0271	1.0420	1.0569	1.0717	48
49	.9361	.9515	.9668	.9821	.9972	1.0123	1.0273	1.0423	1.0571	1.0719	49
50	.9364	.9518	.9671	.9823	.9975	1.0126	1.0276	1.0425	1.0574	1.0721	50
51	.9366	.9520	.9673	.9826	.9977	1.0128	1.0278	1.0428	1.0576	1.0724	51
52	.9369	.9523	.9676	.9828	.9980	1.0131	1.0281	1.0430	1.0579	1.0726	52
53	.9371	.9525	.9678	.9831	.9982	1.0133	1.0283	1.0433	1.0581	1.0729	53
54	.9374	.9528	.9681	.9833	.9985	1.0136	1.0286	1.0435	1.0584	1.0731	54
55	.9377	.9530	.9683	.9836	.9987	1.0138	1.0288	1.0438	1.0586	1.0734	55
56	.9379	.9533	.9686	.9838	.9990	1.0141	1.0291	1.0440	1.0589	1.0736	56
57	.9382	.9536	.9689	.9841	.9992	1.0143	1.0293	1.0443	1.0591	1.0739	57
58	.9384	.9538	.9691	.9843	.9995	1.0146	1.0296	1.0445	1.0593	1.0741	58
59	.9387	.9541	.9694	.9846	.9998	1.0148	1.0298	1.0447	1.0596	1.0744	59
60	.9389	.9543	.9696	.9848	1.0000	1.0151	1.0301	1.0450	1.0598	1.0746	60

Table of Chords; Radius=1.0000 (*continued*).

M.	65°	66°	67°	68°	69°	70°	71°	72°	73°	M.
0′	1.0746	1.0893	1.1039	1.1184	1.1328	1.1472	1.1614	1.1756	1.1896	0′
1	1.0748	1.0895	1.1041	1.1186	1.1331	1.1474	1.1616	1.1758	1.1899	1
2	1.0751	1.0898	1.1044	1.1189	1.1333	1.1476	1.1619	1.1760	1.1901	2
3	1.0753	1.0900	1.1046	1.1191	1.1335	1.1479	1.1621	1.1763	1.1903	3
4	1.0756	1.0903	1.1048	1.1194	1.1338	1.1481	1.1624	1.1765	1.1906	4
5	1.0758	1.0905	1.1051	1.1196	1.1340	1.1483	1.1626	1.1767	1.1908	5
6	1.0761	1.0907	1.1053	1.1198	1.1342	1.1486	1.1628	1.1770	1.1910	6
7	1.0763	1.0910	1.1056	1.1201	1.1345	1.1488	1.1631	1.1772	1.1913	7
8	1.0766	1.0912	1.1058	1.1203	1.1347	1.1491	1.1633	1.1775	1.1915	8
9	1.0768	1.0915	1.1061	1.1206	1.1350	1.1493	1.1635	1.1777	1.1917	9
10	1.0771	1.0917	1.1063	1.1208	1.1352	1.1495	1.1638	1.1779	1.1920	10
11	1.0773	1.0920	1.1065	1.1210	1.1354	1.1498	1.1640	1.1782	1.1922	11
12	1.0775	1.0922	1.1068	1.1213	1.1357	1.1500	1.1642	1.1784	1.1924	12
13	1.0778	1.0924	1.1070	1.1215	1.1359	1.1502	1.1645	1.1786	1.1927	13
14	1.0780	1.0927	1.1073	1.1218	1.1362	1.1505	1.1647	1.1789	1.1929	14
15	1.0783	1.0929	1.1075	1.1220	1.1364	1.1507	1.1650	1.1791	1.1931	15
16	1.0785	1.0932	1.1078	1.1222	1.1366	1.1510	1.1652	1.1793	1.1934	16
17	1.0788	1.0934	1.1080	1.1225	1.1369	1.1512	1.1654	1.1796	1.1936	17
18	1.0790	1.0937	1.1082	1.1227	1.1371	1.1514	1.1657	1.1798	1.1938	18
19	1.0793	1.0939	1.1085	1.1230	1.1374	1.1517	1.1659	1.1800	1.1941	19
20	1.0795	1.0942	1.1087	1.1232	1.1376	1.1519	1.1661	1.1803	1.1943	20
21	1.0797	1.0944	1.1090	1.1234	1.1378	1.1522	1.1664	1.1805	1.1946	21
22	1.0800	1.0946	1.1092	1.1237	1.1381	1.1524	1.1666	1.1807	1.1948	22
23	1.0802	1.0949	1.1094	1.1239	1.1383	1.1526	1.1668	1.1810	1.1950	23
24	1.0805	1.0951	1.1097	1.1242	1.1386	1.1529	1.1671	1.1812	1.1952	24
25	1.0807	1.0954	1.1099	1.1244	1.1388	1.1531	1.1673	1.1814	1.1955	25
26	1.0810	1.0956	1.1102	1.1246	1.1390	1.1533	1.1676	1.1817	1.1957	26
27	1.0812	1.0959	1.1104	1.1249	1.1393	1.1536	1.1678	1.1819	1.1959	27
28	1.0815	1.0961	1.1107	1.1251	1.1395	1.1538	1.1680	1.1821	1.1962	28
29	1.0817	1.0963	1.1109	1.1254	1.1398	1.1541	1.1683	1.1824	1.1964	29
30	1.0820	1.0966	1.1111	1.1256	1.1400	1.1543	1.1685	1.1826	1.1966	30
31	1.0822	1.0968	1.1114	1.1258	1.1402	1.1545	1.1687	1.1829	1.1969	31
32	1.0824	1.0971	1.1116	1.1261	1.1405	1.1548	1.1690	1.1831	1.1971	32
33	1.0827	1.0973	1.1119	1.1263	1.1407	1.1550	1.1692	1.1833	1.1973	33
34	1.0829	1.0976	1.1121	1.1266	1.1409	1.1552	1.1694	1.1836	1.1976	34
35	1.0832	1.0978	1.1123	1.1268	1.1412	1.1555	1.1697	1.1838	1.1978	35
36	1.0834	1.0980	1.1126	1.1271	1.1414	1.1557	1.1699	1.1840	1.1980	36
37	1.0837	1.0983	1.1128	1.1273	1.1417	1.1560	1.1702	1.1843	1.1983	37
38	1.0839	1.0985	1.1131	1.1275	1.1419	1.1562	1.1704	1.1845	1.1985	38
39	1.0841	1.0988	1.1133	1.1278	1.1421	1.1564	1.1706	1.1847	1.1987	39
40	1.0844	1.0990	1.1136	1.1280	1.1424	1.1567	1.1709	1.1850	1.1990	40
41	1.0846	1.0993	1.1138	1.1283	1.1426	1.1569	1.1711	1.1852	1.1992	41
42	1.0849	1.0995	1.1140	1.1285	1.1429	1.1571	1.1713	1.1854	1.1994	42
43	1.0851	1.0997	1.1143	1.1287	1.1431	1.1574	1.1716	1.1857	1.1997	43
44	1.0854	1.1000	1.1145	1.1290	1.1433	1.1576	1.1718	1.1859	1.1999	44
45	1.0856	1.1002	1.1148	1.1292	1.1436	1.1579	1.1720	1.1861	1.2001	45
46	1.0859	1.1005	1.1150	1.1295	1.1438	1.1581	1.1723	1.1864	1.2004	46
47	1.0861	1.1007	1.1152	1.1297	1.1441	1.1583	1.1725	1.1866	1.2006	47
48	1.0863	1.1010	1.1155	1.1299	1.1443	1.1586	1.1727	1.1868	1.2008	48
49	1.0866	1.1012	1.1157	1.1302	1.1445	1.1588	1.1730	1.1871	1.2011	49
50	1.0868	1.1014	1.1160	1.1304	1.1448	1.1590	1.1732	1.1873	1.2013	50
51	1.0871	1.1017	1.1162	1.1307	1.1450	1.1593	1.1735	1.1875	1.2015	51
52	1.0873	1.1019	1.1165	1.1309	1.1452	1.1595	1.1737	1.1878	1.2018	52
53	1.0876	1.1022	1.1167	1.1311	1.1455	1.1598	1.1739	1.1880	1.2020	53
54	1.0878	1.1024	1.1169	1.1314	1.1457	1.1600	1.1742	1.1882	1.2022	54
55	1.0881	1.1027	1.1172	1.1316	1.1460	1.1602	1.1744	1.1885	1.2025	55
56	1.0883	1.1029	1.1174	1.1319	1.1462	1.1605	1.1746	1.1887	1.2027	56
57	1.0885	1.1031	1.1177	1.1321	1.1464	1.1607	1.1749	1.1889	1.2029	57
58	1.0888	1.1034	1.1179	1.1323	1.1467	1.1609	1.1751	1.1892	1.2032	58
59	1.0890	1.1036	1.1181	1.1326	1.1469	1.1612	1.1753	1.1894	1.2034	59
60	1.0893	1.1039	1.1184	1.1328	1.1472	1.1614	1.1756	1.1896	1.2036	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	74°	75°	76°	77°	78°	79°	80°	81°	82°	M.
0′	1.2036	1.2175	1.2313	1.2450	1.2586	1.2722	1.2856	1.2989	1.3121	0′
1	1.2039	1.2178	1.2316	1.2453	1.2589	1.2724	1.2858	1.2991	1.3123	1
2	1.2041	1.2180	1.2318	1.2455	1.2591	1.2726	1.2860	1.2993	1.3126	2
3	1.2043	1.2182	1.2320	1.2457	1.2593	1.2728	1.2862	1.2996	1.3128	3
4	1.2046	1.2184	1.2322	1.2459	1.2595	1.2731	1.2865	1.2998	1.3130	4
5	1.2048	1.2187	1.2325	1.2462	1.2598	1.2733	1.2867	1.3000	1.3132	5
6	1.2050	1.2189	1.2327	1.2464	1.2600	1.2735	1.2869	1.3002	1.3134	6
7	1.2053	1.2191	1.2329	1.2466	1.2602	1.2737	1.2871	1.3004	1.3137	7
8	1.2055	1.2194	1.2332	1.2468	1.2604	1.2740	1.2874	1.3007	1.3139	8
9	1.2057	1.2196	1.2334	1.2471	1.2607	1.2742	1.2876	1.3009	1.3141	9
10	1.2060	1.2198	1.2336	1.2473	1.2609	1.2744	1.2878	1.3011	1.3143	10
11	1.2062	1.2201	1.2338	1.2475	1.2611	1.2746	1.2880	1.3013	1.3145	11
12	1.2064	1.2203	1.2341	1.2478	1.2614	1.2748	1.2882	1.3015	1.3147	12
13	1.2066	1.2205	1.2343	1.2480	1.2616	1.2751	1.2885	1.3018	1.3150	13
14	1.2069	1.2208	1.2345	1.2482	1.2618	1.2753	1.2887	1.3020	1.3152	14
15	1.2071	1.2210	1.2348	1.2484	1.2620	1.2755	1.2889	1.3022	1.3154	15
16	1.2073	1.2212	1.2350	1.2487	1.2623	1.2757	1.2891	1.3024	1.3156	16
17	1.2076	1.2214	1.2352	1.2489	1.2625	1.2760	1.2894	1.3027	1.3158	17
18	1.2078	1.2217	1.2354	1.2491	1.2627	1.2762	1.2896	1.3029	1.3161	18
19	1.2080	1.2219	1.2357	1.2493	1.2629	1.2764	1.2898	1.3031	1.3163	19
20	1.2083	1.2221	1.2359	1.2496	1.2632	1.2766	1.2900	1.3033	1.3165	20
21	1.2085	1.2224	1.2361	1.2498	1.2634	1.2769	1.2903	1.3035	1.3167	21
22	1.2087	1.2226	1.2364	1.2500	1.2636	1.2771	1.2905	1.3038	1.3169	22
23	1.2090	1.2228	1.2366	1.2503	1.2638	1.2773	1.2907	1.3040	1.3172	23
24	1.2092	1.2231	1.2368	1.2505	1.2641	1.2775	1.2909	1.3042	1.3174	24
25	1.2094	1.2233	1.2370	1.2507	1.2643	1.2778	1.2911	1.3044	1.3176	25
26	1.2097	1.2235	1.2373	1.2509	1.2645	1.2780	1.2914	1.3046	1.3178	26
27	1.2099	1.2237	1.2375	1.2512	1.2648	1.2782	1.2916	1.3049	1.3180	27
28	1.2101	1.2240	1.2377	1.2514	1.2650	1.2784	1.2918	1.3051	1.3183	28
29	1.2104	1.2242	1.2380	1.2516	1.2652	1.2787	1.2920	1.3053	1.3185	29
30	1.2106	1.2244	1.2382	1.2518	1.2654	1.2789	1.2922	1.3055	1.3187	30
31	1.2108	1.2247	1.2384	1.2521	1.2656	1.2791	1.2925	1.3057	1.3189	31
32	1.2111	1.2249	1.2386	1.2523	1.2659	1.2793	1.2927	1.3060	1.3191	32
33	1.2113	1.2251	1.2389	1.2525	1.2661	1.2795	1.2929	1.3062	1.3193	33
34	1.2115	1.2254	1.2391	1.2528	1.2663	1.2798	1.2931	1.3064	1.3196	34
35	1.2117	1.2256	1.2393	1.2530	1.2665	1.2800	1.2934	1.3066	1.3198	35
36	1.2120	1.2258	1.2396	1.2532	1.2668	1.2802	1.2936	1.3068	1.3200	36
37	1.2122	1.2260	1.2398	1.2534	1.2670	1.2804	1.2938	1.3071	1.3202	37
38	1.2124	1.2263	1.2400	1.2537	1.2672	1.2807	1.2940	1.3073	1.3204	38
39	1.2127	1.2265	1.2402	1.2539	1.2674	1.2809	1.2942	1.3075	1.3207	39
40	1.2129	1.2267	1.2405	1.2541	1.2677	1.2811	1.2945	1.3077	1.3209	40
41	1.2131	1.2270	1.2407	1.2543	1.2679	1.2813	1.2947	1.3079	1.3211	41
42	1.2134	1.2272	1.2409	1.2546	1.2681	1.2816	1.2949	1.3082	1.3213	42
43	1.2136	1.2274	1.2412	1.2548	1.2683	1.2818	1.2951	1.3084	1.3215	43
44	1.2138	1.2277	1.2414	1.2550	1.2686	1.2820	1.2954	1.3086	1.3218	44
45	1.2141	1.2279	1.2416	1.2552	1.2688	1.2822	1.2956	1.3088	1.3220	45
46	1.2143	1.2281	1.2418	1.2555	1.2690	1.2825	1.2958	1.3090	1.3222	46
47	1.2145	1.2283	1.2421	1.2557	1.2692	1.2827	1.2960	1.3093	1.3224	47
48	1.2148	1.2286	1.2423	1.2559	1.2695	1.2829	1.2962	1.3095	1.3226	48
49	1.2150	1.2288	1.2425	1.2562	1.2697	1.2831	1.2965	1.3097	1.3228	49
50	1.2152	1.2290	1.2428	1.2564	1.2699	1.2833	1.2967	1.3099	1.3231	50
51	1.2154	1.2293	1.2430	1.2566	1.2701	1.2836	1.2969	1.3101	1.3233	51
52	1.2157	1.2295	1.2432	1.2568	1.2704	1.2838	1.2971	1.3104	1.3235	52
53	1.2159	1.2297	1.2434	1.2571	1.2706	1.2840	1.2973	1.3106	1.3237	53
54	1.2161	1.2299	1.2437	1.2573	1.2708	1.2842	1.2976	1.3108	1.3239	54
55	1.2164	1.2302	1.2439	1.2575	1.2710	1.2845	1.2978	1.3110	1.3242	55
56	1.2166	1.2304	1.2441	1.2577	1.2713	1.2847	1.2980	1.3112	1.3244	56
57	1.2168	1.2306	1.2443	1.2580	1.2715	1.2849	1.2982	1.3115	1.3246	57
58	1.2171	1.2309	1.2446	1.2582	1.2717	1.2851	1.2985	1.3117	1.3248	58
59	1.2173	1.2311	1.2448	1.2584	1.2719	1.2854	1.2987	1.3119	1.3250	59
60	1.2175	1.2313	1.2450	1.2586	1.2722	1.2856	1.2989	1.3121	1.3252	60

Table of Chords; Radius = 1.0000 (*concluded*).

M.	83°	84°	85°	86°	87°	88°	89°	M.
0′	1.3252	1.3383	1.3512	1.3640	1.3767	1.3893	1.4018	0′
1	1.3255	1.3385	1.3514	1.3642	1.3769	1.3895	1.4020	1
2	1.3257	1.3387	1.3516	1.3644	1.3771	1.3897	1.4022	2
3	1.3259	1.3389	1.3518	1.3646	1.3773	1.3899	1.4024	3
4	1.3261	1.3391	1.3520	1.3648	1.3776	1.3902	1.4026	4
5	1.3263	1.3393	1.3523	1.3651	1.3778	1.3904	1.4029	5
6	1.3265	1.3396	1.3525	1.3653	1.3780	1.3906	1.4031	6
7	1.3268	1.3398	1.3527	1.3655	1.3782	1.3908	1.4033	7
8	1.3270	1.3400	1.3529	1.3657	1.3784	1.3910	1.4035	8
9	1.3272	1.3402	1.3531	1.3659	1.3786	1.3912	1.4037	9
10	1.3274	1.3404	1.3533	1.3661	1.3788	1.3914	1.4039	10
11	1.3276	1.3406	1.3535	1.3663	1.3790	1.3916	1.4041	11
12	1.3279	1.3409	1.3538	1.3665	1.3792	1.3918	1.4043	12
13	1.3281	1.3411	1.3540	1.3668	1.3794	1.3920	1.4045	13
14	1.3283	1.3413	1.3542	1.3670	1.3797	1.3922	1.4047	14
15	1.3285	1.3415	1.3544	1.3672	1.3799	1.3925	1.4049	15
16	1.3287	1.3417	1.3546	1.3674	1.3801	1.3927	1.4051	16
17	1.3289	1.3419	1.3548	1.3676	1.3803	1.3929	1.4053	17
18	1.3292	1.3421	1.3550	1.3678	1.3805	1.3931	1.4055	18
19	1.3294	1.3424	1.3552	1.3680	1.3807	1.3933	1.4058	19
20	1.3296	1.3426	1.3555	1.3682	1.3809	1.3935	1.4060	20
21	1.3298	1.3428	1.3557	1.3685	1.3811	1.3937	1.4062	21
22	1.3300	1.3430	1.3559	1.3687	1.3813	1.3939	1.4064	22
23	1.3302	1.3432	1.3561	1.3689	1.3816	1.3941	1.4066	23
24	1.3305	1.3434	1.3563	1.3691	1.3818	1.3943	1.4068	24
25	1.3307	1.3437	1.3565	1.3693	1.3820	1.3945	1.4070	25
26	1.3309	1.3439	1.3567	1.3695	1.3822	1.3947	1.4072	26
27	1.3311	1.3441	1.3570	1.3697	1.3824	1.3950	1.4074	27
28	1.3313	1.3443	1.3572	1.3699	1.3826	1.3952	1.4076	28
29	1.3315	1.3445	1.3574	1.3702	1.3828	1.3954	1.4078	29
30	1.3318	1.3447	1.3576	1.3704	1.3830	1.3956	1.4080	30
31	1.3320	1.3449	1.3578	1.3706	1.3832	1.3958	1.4082	31
32	1.3322	1.3452	1.3580	1.3708	1.3834	1.3960	1.4084	32
33	1.3324	1.3454	1.3582	1.3710	1.3837	1.3962	1.4086	33
34	1.3326	1.3456	1.3585	1.3712	1.3839	1.3964	1.4089	34
35	1.3328	1.3458	1.3587	1.3714	1.3841	1.3966	1.4091	35
36	1.3331	1.3460	1.3589	1.3716	1.3843	1.3968	1.4093	36
37	1.3333	1.3462	1.3591	1.3718	1.3845	1.3970	1.4095	37
38	1.3335	1.3465	1.3593	1.3721	1.3847	1.3972	1.4097	38
39	1.3337	1.3467	1.3595	1.3723	1.3849	1.3975	1.4099	39
40	1.3339	1.3469	1.3597	1.3725	1.3851	1.3977	1.4101	40
41	1.3341	1.3471	1.3599	1.3727	1.3853	1.3979	1.4103	41
42	1.3344	1.3473	1.3602	1.3729	1.3855	1.3981	1.4105	42
43	1.3346	1.3475	1.3604	1.3731	1.3858	1.3983	1.4107	43
44	1.3348	1.3477	1.3606	1.3733	1.3860	1.3985	1.4109	44
45	1.3350	1.3480	1.3608	1.3735	1.3862	1.3987	1.4111	45
46	1.3352	1.3482	1.3610	1.3738	1.3864	1.3989	1.4113	46
47	1.3354	1.3484	1.3612	1.3740	1.3866	1.3991	1.4115	47
48	1.3357	1.3486	1.3614	1.3742	1.3868	1.3993	1.4117	48
49	1.3359	1.3488	1.3617	1.3744	1.3870	1.3995	1.4119	49
50	1.3361	1.3490	1.3619	1.3746	1.3872	1.3997	1.4122	50
51	1.3363	1.3492	1.3621	1.3748	1.3874	1.3999	1.4124	51
52	1.3365	1.3495	1.3623	1.3750	1.3876	1.4002	1.4126	52
53	1.3367	1.3497	1.3625	1.3752	1.3879	1.4004	1.4128	53
54	1.3370	1.3499	1.3627	1.3754	1.3881	1.4006	1.4130	54
55	1.3372	1.3501	1.3629	1.3757	1.3883	1.4008	1.4132	55
56	1.3374	1.3503	1.3631	1.3759	1.3885	1.4010	1.4134	56
57	1.3376	1.3505	1.3634	1.3761	1.3887	1.4012	1.4136	57
58	1.3378	1.3508	1.3636	1.3763	1.3889	1.4014	1.4138	58
59	1.3380	1.3510	1.3638	1.3765	1.3891	1.4016	1.4140	59
60	1.3383	1.3512	1.3640	1.3767	1.3893	1.4018	1.4142	60

Lengths and Bevels of Hip and Jack Rafters.

The lines *ab* and *bc* in Fig. 89 represent the walls at the angle of a building; *be* is the seat of the hip-rafter, and *gf* of a jack-rafter. Draw *eh* at right angles to *be*, and make it equal to the rise of the roof; join *b* and *h*, and *hb* will be the length of the hip-rafter. Through *e* draw *di* at right angles to *bc*. Upon *b*, with the radius *bh*, describe the arc *hi*, cutting *di* in *i*. Join *b* and *i*,

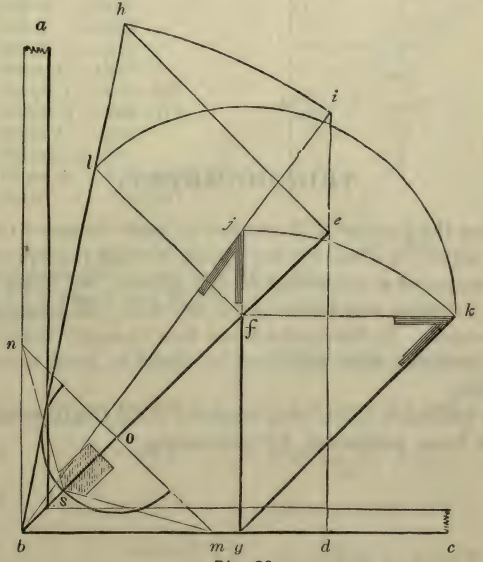


Fig. 89.

and extend *gf* to meet *bi* in *j*; then *gj* will be the length of the jack-rafter. The length of each jack-rafter is found in the same manner,—by extending its seat to cut the line *bi*. From *f* draw *fk* at right angles to *fg*, also *fl* at right angles to *be*. Make *fk* equal to *fl* by the arc *lk*, or make *gk* equal to *gj* by the arc *jk*; then the angle at *j* will be the *top bevel* of the jack-rafters, and the one at *k* the *down bevel*.

Backing of the hip-rafter. At any convenient place in *be* (Fig. 89), as *o*, draw *mn* at right angles to *be*. From *o* describe a circle, tangent to *bh*, cutting *be* in *s*. Join *m* and *s* and *n* and *s*; then these lines will form at *s* the proper angle for bevelling the top of the hip-rafter.

TRIGONOMETRY.

It is not the purpose of the author to teach the use of trigonometry, or what it is; but, for the benefit of those readers who have already acquired a knowledge of this science, the following convenient formulas, and tables of natural sines and tangents, have been inserted. To those who know how to apply these trigonometric functions, they will often be found of great convenience and utility.

These tables are taken from Searle's "Field Engineering," John Wiley & Sons, publishers, by permission.

TRIGONOMETRIC FUNCTIONS.

Let A (Fig. 107) = angle BAC = arc BF, and let the radius $AF = AB = AH = 1$.

We then have

$\sin A = BC$
$\cos A = AC$
$\tan A = DF$
$\cot A = HG$
$\sec A = AD$
$\operatorname{cosec} A = AG$
$\operatorname{versin} A = CF = BE$
$\operatorname{covers} A = BK = HL$
$\operatorname{exsec} A = BD$
$\operatorname{coexsec} A = BG$
$\operatorname{chord} A = BF$
$\operatorname{chord} 2A = BI = 2BC$

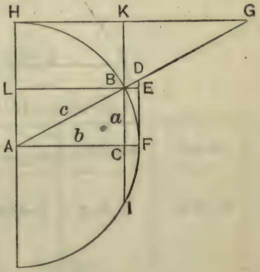


FIG. 107.

In the right-angled triangle ABC (Fig. 107)
Let $AB = c$, $AC = b$, and $BC = a$.
We then have:

1. $\sin A = \dfrac{a}{c} = \cos B$
2. $\cos A = \dfrac{b}{c} = \sin B$
3. $\tan A = \dfrac{a}{b} = \cot B$
4. $\cot A = \dfrac{b}{a} = \tan B$
5. $\sec A = \dfrac{c}{b} = \operatorname{cosec} B$
6. $\operatorname{cosec} A = \dfrac{c}{a} = \sec B$
7. $\operatorname{vers} A = \dfrac{c-b}{c} = \operatorname{covers} B$
8. $\operatorname{exsec} A = \dfrac{c-b}{b} = \operatorname{coexsec} B$
9. $\operatorname{covers} A = \dfrac{c-a}{c} = \operatorname{versin} B$
10. $\operatorname{coexsec} A = \dfrac{c-a}{a} = \operatorname{exsec} B$
11. $a = c \sin A = b \tan A$
12. $b = c \cos A = a \cot A$
13. $c = \dfrac{a}{\sin A} = \dfrac{b}{\cos A}$
14. $a = c \cos B = b \cot B$
15. $b = c \sin B = a \tan B$
16. $c = \dfrac{a}{\cos B} = \dfrac{b}{\sin B}$
17. $a = \sqrt{(c+b)(c-b)}$
18. $b = \sqrt{(c+a)(c-a)}$
19. $c = \sqrt{a^2+b^2}$
20. $C = 90° = A + B$

21. $\text{area} = \dfrac{ab}{2}$

Solution of Oblique Triangles.

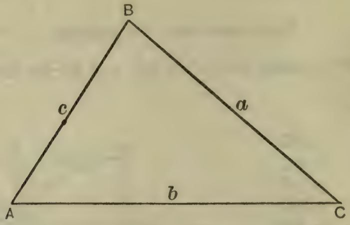


Fig. 108.

	GIVEN.	SOUGHT.	FORMULÆ.
22	A, B, a	C, b, c	$C=180^\circ-(A+B)$, $b=\frac{a}{\sin A}.\sin B$, $c=\frac{a}{\sin A}\sin(A+B)$
23	A, a, b	B, C, c	$\sin B=\frac{\sin A}{a}.b$, $C=180^\circ-(A+B)$, $c=\frac{a}{\sin A}.\sin C$.
24	C, a, b	$\frac{1}{2}(A+B)$	$\frac{1}{2}(A+B)=90^\circ-\frac{1}{2}C$
25		$\frac{1}{2}(A-B)$	$\tan\frac{1}{2}(A-B)=\frac{a-b}{a+b}\tan\frac{1}{2}(A+B)$
26		A, B	$A=\frac{1}{2}(A+B)+\frac{1}{2}(A-B)$, $B=\frac{1}{2}(A+B)-\frac{1}{2}(A-B)$
27		c	$c=(a+b)\frac{\cos\frac{1}{2}(A+B)}{\cos\frac{1}{2}(A-B)}=(a-b)\frac{\sin\frac{1}{2}(A+B)}{\sin\frac{1}{2}(A-B)}$
28		area	$K=\frac{1}{2}ab\sin C$.
29	a, b, c	A	Let $s=\frac{1}{2}(a+b+c)$; $\sin\frac{1}{2}A=\sqrt{\frac{(s-b)(s-c)}{bc}}$
30			$\cos\frac{1}{2}A=\sqrt{\frac{s(s-a)}{bc}}$; $\tan\frac{1}{2}A=\sqrt{\frac{(s-b)(s-c)}{s(s-a)}}$
31			$\sin A=\frac{2\sqrt{s(s-a)(s-b)(s-c)}}{bc}$; $\operatorname{vers} A=\frac{2(s-b)(s-c)}{bc}$
32		area	$K=\sqrt{s(s-a)(s-b)(s-c)}$
33	A, B, C, a	area	$K=\frac{a^2\sin B.\sin C}{2\sin A}$

General Formulæ.

34. $\sin A = \dfrac{1}{\operatorname{cosec} A} = \sqrt{1-\cos^2 A} = \tan A \cos A$

35. $\sin A = 2 \sin \tfrac{1}{2}A \cos \tfrac{1}{2}A = \operatorname{vers} A \cot \tfrac{1}{2}A$

36. $\sin A = \sqrt{\tfrac{1}{2}\operatorname{vers} 2A} = \sqrt{\tfrac{1}{2}(1-\cos 2A)}$

37. $\cos A = \dfrac{1}{\sec A} = \sqrt{1-\sin^2 A} = \cot A \sin A$

38. $\cos A = 1 - \operatorname{vers} A = 2 \cos^2 \tfrac{1}{2}A - 1 = 1 - 2 \sin^2 \tfrac{1}{2}A$

39. $\cos A = \cos^2 \tfrac{1}{2}A - \sin^2 \tfrac{1}{2}A = \sqrt{\tfrac{1}{2} + \tfrac{1}{2}\cos 2A}$

40. $\tan A = \dfrac{1}{\cot A} = \dfrac{\sin A}{\cos A} = \sqrt{\sec^2 A - 1}$

41. $\tan A = \sqrt{\dfrac{1}{\cos^2 A} - 1} = \dfrac{\sqrt{1-\cos^2 A}}{\cos A} = \dfrac{\sin 2A}{1+\cos 2A}$

42. $\tan A = \dfrac{1-\cos 2A}{\sin 2A} = \dfrac{\operatorname{vers} 2A}{\sin 2A} = \operatorname{exsec} A \cot \tfrac{1}{2}A$

43. $\cot A = \dfrac{1}{\tan A} = \dfrac{\cos A}{\sin A} = \sqrt{\operatorname{cosec}^2 A - 1}$

44. $\cot A = \dfrac{\sin 2A}{1-\cos 2A} = \dfrac{\sin 2A}{\operatorname{vers} 2A} = \dfrac{1+\cos 2A}{\sin 2A}$

45. $\cot A = \dfrac{\tan \tfrac{1}{2}A}{\operatorname{exsec} A}$

46. $\operatorname{vers} A = 1 - \cos A = \sin A \tan \tfrac{1}{2}A = 2 \sin^2 \tfrac{1}{2}A$

47. $\operatorname{vers} A = \operatorname{exsec} A \cos A$

48. $\operatorname{exsec} A = \sec A - 1 = \tan A \tan \tfrac{1}{2}A = \dfrac{\operatorname{vers} A}{\cos A}$

49. $\sin \tfrac{1}{2}A = \sqrt{\dfrac{1-\cos A}{2}} = \sqrt{\dfrac{\operatorname{vers} A}{2}}$

50. $\sin 2A = 2 \sin A \cos A$

51. $\cos \tfrac{1}{2}A = \sqrt{\dfrac{1+\cos A}{2}}$

52. $\cos 2A = 2 \cos^2 A - 1 = \cos^2 A - \sin^2 A = 1 - 2 \sin^2 A$

General Formulæ.

53. $\tan \frac{1}{2}A = \dfrac{\tan A}{1+\sec A} = \operatorname{cosec} A - \cot A = \dfrac{1-\cos A}{\sin A} = \sqrt{\dfrac{1-\cos A}{1+\cos A}}$

54. $\tan 2A = \dfrac{2 \tan A}{1-\tan^2 A}$

55. $\cot \frac{1}{2}A = \dfrac{\sin A}{\operatorname{vers} A} = \dfrac{1+\cos A}{\sin A} = \dfrac{1}{\operatorname{cosec} A - \cot A}$

56. $\cot 2A = \dfrac{\cot^2 A - 1}{2 \cot A}$

57. $\operatorname{vers} \frac{1}{2}A = \dfrac{\frac{1}{2} \operatorname{vers} A}{1+\sqrt{1-\frac{1}{2} \operatorname{vers} A}} = \dfrac{1-\cos A}{2+\sqrt{2(1+\cos A)}}$

58. $\operatorname{vers} 2A = 2 \sin^2 A$

59. $\operatorname{exsec} \frac{1}{2}A = \dfrac{1-\cos A}{(1+\cos A)+\sqrt{2(1+\cos A)}}$

60. $\operatorname{exsec} 2A = \dfrac{2 \tan^2 A}{1-\tan^2 A}$

61. $\sin(A \pm B) = \sin A . \cos B \pm \sin B . \cos A$

62. $\cos (A \pm B) = \cos A . \cos B \mp \sin A . \sin B$

63. $\sin A + \sin B = 2 \sin \frac{1}{2}(A+B) \cos \frac{1}{2}(A-B)$

64. $\sin A - \sin B = 2 \cos \frac{1}{2}(A+B) \sin \frac{1}{2}(A-B)$

65. $\cos A + \cos B = 2 \cos \frac{1}{2}(A+B) \cos \frac{1}{2}(A-B)$

66. $\cos B - \cos A = 2 \sin \frac{1}{2}(A+B) \sin \frac{1}{2}(A-B)$

67. $\sin^2 A - \sin^2 B = \cos^2 B - \cos^2 A = \sin (A+B) \sin (A-B)$

68. $\cos^2 A - \sin^2 B = \cos(A+B) \cos (A-B)$

69. $\tan A + \tan B = \dfrac{\sin (A+B)}{\cos A . \cos B}$

70. $\tan A - \tan B = \dfrac{\sin (A-B)}{\cos A . \cos B}$

′	0° Sine	0° Cosin	1° Sine	1° Cosin	2° Sine	2° Cosin	3° Sine	3° Cosin	4° Sine	4° Cosin	′
0	.00000	One.	.01745	.99985	.03490	.99939	.05234	.99863	.06976	.99756	60
1	.00029	One.	.01774	.99984	.03519	.99938	.05263	.99861	.07005	.99754	59
2	.00058	One.	.01803	.99984	.03548	.99937	.05292	.99860	.07034	.99752	58
3	.00087	One.	.01832	.99983	.03577	.99936	.05321	.99858	.07063	.99750	57
4	.00116	One.	.01862	.99983	.03606	.99935	.05350	.99857	.07092	.99748	56
5	.00145	One.	.01891	.99982	.03635	.99934	.05379	.99855	.07121	.99746	55
6	.00175	One.	.01920	.99982	.03664	.99933	.05408	.99854	.07150	.99744	54
7	.00204	One.	.01949	.99981	.03693	.99932	.05437	.99852	.07179	.99742	53
8	.00233	One.	.01978	.99980	.03723	.99931	.05466	.99851	.07208	.99740	52
9	.00262	One.	.02007	.99980	.03752	.99930	.05495	.99849	.07237	.99738	51
10	.00291	One.	.02036	.99979	.03781	.99929	.05524	.99847	.07266	.99736	50
11	.00320	.99999	.02065	.99979	.03810	.99927	.05553	.99846	.07295	.99734	49
12	.00349	.99999	.02094	.99978	.03839	.99926	.05582	.99844	.07324	.99731	48
13	.00378	.99999	.02123	.99977	.03868	.99925	.05611	.99842	.07353	.99729	47
14	.00407	.99999	.02152	.99977	.03897	.99924	.05640	.99841	.07382	.99727	46
15	.00436	.99999	.02181	.99976	.03926	.99923	.05669	.99839	.07411	.99725	45
16	.00465	.99999	.02211	.99976	.03955	.99922	.05698	.99838	.07440	.99723	44
17	.00495	.99999	.02240	.99975	.03984	.99921	.05727	.99836	.07469	.99721	43
18	.00524	.99999	.02269	.99974	.04013	.99919	.05756	.99834	.07498	.99719	42
19	.00553	.99998	.02298	.99974	.04042	.99918	.05785	.99833	.07527	.99716	41
20	.00582	.99998	.02327	.99973	.04071	.99917	.05814	.99831	.07556	.99714	40
21	.00611	.99998	.02356	.99972	.04100	.99916	.05844	.99829	.07585	.99712	39
22	.00640	.99998	.02385	.99972	.04129	.99915	.05873	.99827	.07614	.99710	38
23	.00669	.99998	.02414	.99971	.04159	.99913	.05902	.99826	.07643	.99708	37
24	.00698	.99998	.02443	.99970	.04188	.99912	.05931	.99824	.07672	.99705	36
25	.00727	.99997	.02472	.99969	.04217	.99911	.05960	.99822	.07701	.99703	35
26	.00756	.99997	.02501	.99969	.04246	.99910	.05989	.99821	.07730	.99701	34
27	.00785	.99997	.02530	.99968	.04275	.99909	.06018	.99819	.07759	.99699	33
28	.00814	.99997	.02560	.99967	.04304	.99907	.06047	.99817	.07788	.99696	32
29	.00844	.99996	.02589	.99966	.04333	.99906	.06076	.99815	.07817	.99694	31
30	.00873	.99996	.02618	.99966	.04362	.99905	.06105	.99813	.07846	.99692	30
31	.00902	.99996	.02647	.99965	.04391	.99904	.06134	.99812	.07875	.99689	29
32	.00931	.99996	.02676	.99964	.04420	.99902	.06163	.99810	.07904	.99687	28
33	.00960	.99995	.02705	.99963	.04449	.99901	.06192	.99808	.07933	.99685	27
34	.00989	.99995	.02734	.99963	.04478	.99900	.06221	.99806	.07962	.99683	26
35	.01018	.99995	.02763	.99962	.04507	.99898	.06250	.99804	.07991	.99680	25
36	.01047	.99995	.02792	.99961	.04536	.99897	.06279	.99803	.08020	.99678	24
37	.01076	.99994	.02821	.99960	.04565	.99896	.06308	.99801	.08049	.99676	23
38	.01105	.99994	.02850	.99959	.04594	.99894	.06337	.99799	.08078	.99673	22
39	.01134	.99994	.02879	.99959	.04623	.99893	.06366	.99797	.08107	.99671	21
40	.01164	.99993	.02908	.99958	.04653	.99892	.06395	.99795	.08136	.99668	20
41	.01193	.99993	.02938	.99957	.04682	.99890	.06424	.99793	.08165	.99666	19
42	.01222	.99993	.02967	.99956	.04711	.99889	.06453	.99792	.08194	.99664	18
43	.01251	.99992	.02996	.99955	.04740	.99888	.06482	.99790	.08223	.99661	17
44	.01280	.99992	.03025	.99954	.04769	.99886	.06511	.99788	.08252	.99659	16
45	.01309	.99991	.03054	.99953	.04798	.99885	.06540	.99786	.08281	.99657	15
46	.01338	.99991	.03083	.99952	.04827	.99883	.06569	.99784	.08310	.99654	14
47	.01367	.99991	.03112	.99952	.04856	.99882	.06598	.99782	.08339	.99652	13
48	.01396	.99990	.03141	.99951	.04885	.99881	.06627	.99780	.08368	.99649	12
49	.01425	.99990	.03170	.99950	.04914	.99879	.06656	.99778	.08397	.99647	11
50	.01454	.99989	.03199	.99949	.04943	.99878	.06685	.99776	.08426	.99644	10
51	.01483	.99989	.03228	.99948	.04972	.99876	.06714	.99774	.08455	.99642	9
52	.01513	.99989	.03257	.99947	.05001	.99875	.06743	.99772	.08484	.99639	8
53	.01542	.99988	.03286	.99946	.05030	.99873	.06773	.99770	.08513	.99637	7
54	.01571	.99988	.03316	.99945	.05059	.99872	.06802	.99768	.08542	.99635	6
55	.01600	.99987	.03345	.99944	.05088	.99870	.06831	.99766	.08571	.99632	5
56	.01629	.99987	.03374	.99943	.05117	.99869	.06860	.99764	.08600	.99630	4
57	.01658	.99986	.03403	.99942	.05146	.99867	.06889	.99762	.08629	.99627	3
58	.01687	.99986	.03432	.99941	.05175	.99866	.06918	.99760	.08658	.99625	2
59	.01716	.99985	.03461	.99940	.05205	.99864	.06947	.99758	.08687	.99622	1
60	.01745	.99985	.03490	.99939	.05234	.99863	.06976	.99756	.08716	.99619	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	89°		88°		87°		86°		85°		

′	5°		6°		7°		8°		9°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.08716	.99619	.10453	.99452	.12187	.99255	.13917	.99027	.15643	.98769	60
1	.08745	.99617	.10482	.99449	.12216	.99251	.13946	.99023	.15672	.98764	59
2	.08774	.99614	.10511	.99446	.12245	.99248	.13975	.99019	.15701	.98760	58
3	.08803	.99612	.10540	.99443	.12274	.99244	.14004	.99015	.15730	.98755	57
4	.08831	.99609	.10569	.99440	.12302	.99240	.14033	.99011	.15758	.98751	56
5	.08860	.99607	.10597	.99437	.12331	.99237	.14061	.99006	.15787	.98746	55
6	.08889	.99604	.10626	.99434	.12360	.99233	.14090	.99002	.15816	.98741	54
7	.08918	.99602	.10655	.99431	.12389	.99230	.14119	.98998	.15845	.98737	53
8	.08947	.99599	.10684	.99428	.12418	.99226	.14148	.98994	.15873	.98732	52
9	.08976	.99596	.10713	.99424	.12447	.99222	.14177	.98990	.15902	.98728	51
10	.09005	.99594	.10742	.99421	.12476	.99219	.14205	.98986	.15931	.98723	50
11	.09034	.99591	.10771	.99418	.12504	.99215	.14234	.98982	.15959	.98718	49
12	.09063	.99588	.10800	.99415	.12533	.99211	.14263	.98978	.15988	.98714	48
13	.09092	.99586	.10829	.99412	.12562	.99208	.14292	.98973	.16017	.98709	47
14	.09121	.99583	.10858	.99409	.12591	.99204	.14320	.98969	.16046	.98704	46
15	.09150	.99580	.10887	.99406	.12620	.99200	.14349	.98965	.16074	.98700	45
16	.09179	.99578	.10916	.99402	.12649	.99197	.14378	.98961	.16103	.98695	44
17	.09208	.99575	.10945	.99399	.12678	.99193	.14407	.98957	.16132	.98690	43
18	.09237	.99572	.10973	.99396	.12706	.99189	.14436	.98953	.16160	.98686	42
19	.09266	.99570	.11002	.99393	.12735	.99186	.14464	.98948	.16189	.98681	41
20	.09295	.99567	.11031	.99390	.12764	.99182	.14493	.98944	.16218	.98676	40
21	.09324	.99564	.11060	.99386	.12793	.99178	.14522	.98940	.16246	.98671	39
22	.09353	.99562	.11089	.99383	.12822	.99175	.14551	.98936	.16275	.98667	38
23	.09382	.99559	.11118	.99380	.12851	.99171	.14580	.98931	.16304	.98662	37
24	.09411	.99556	.11147	.99377	.12880	.99167	.14608	.98927	.16333	.98657	36
25	.09440	.99553	.11176	.99374	.12908	.99163	.14637	.98923	.16361	.98652	35
26	.09469	.99551	.11205	.99370	.12937	.99160	.14666	.98919	.16390	.98648	34
27	.09498	.99548	.11234	.99367	.12966	.99156	.14695	.98914	.16419	.98643	33
28	.09527	.99545	.11263	.99364	.12995	.99152	.14723	.98910	.16447	.98638	32
29	.09556	.99542	.11291	.99360	.13024	.99148	.14752	.98906	.16476	.98633	31
30	.09585	.99540	.11320	.99357	.13053	.99144	.14781	.98902	.16505	.98629	30
31	.09614	.99537	.11349	.99354	.13081	.99141	.14810	.98897	.16533	.98624	29
32	.09642	.99534	.11378	.99351	.13110	.99137	.14838	.98893	.16562	.98619	28
33	.09671	.99531	.11407	.99347	.13139	.99133	.14867	.98889	.16591	.98614	27
34	.09700	.99528	.11436	.99344	.13168	.99129	.14896	.98884	.16620	.98609	26
35	.09729	.99526	.11465	.99341	.13197	.99125	.14925	.98880	.16648	.98604	25
36	.09758	.99523	.11494	.99337	.13226	.99122	.14954	.98876	.16677	.98600	24
37	.09787	.99520	.11523	.99334	.13254	.99118	.14982	.98871	.16706	.98595	23
38	.09816	.99517	.11552	.99331	.13283	.99114	.15011	.98867	.16734	.98590	22
39	.09845	.99514	.11580	.99327	.13312	.99110	.15040	.98863	.16763	.98585	21
40	.09874	.99511	.11609	.99324	.13341	.99106	.15069	.98858	.16792	.98580	20
41	.09903	.99508	.11638	.99320	.13370	.99102	.15097	.98854	.16820	.98575	19
42	.09932	.99506	.11667	.99317	.13399	.99098	.15126	.98849	.16849	.98570	18
43	.09961	.99503	.11696	.99314	.13427	.99094	.15155	.98845	.16878	.98565	17
44	.09990	.99500	.11725	.99310	.13456	.99091	.15184	.98841	.16906	.98561	16
45	.10019	.99497	.11754	.99307	.13485	.99087	.15212	.98836	.16935	.98556	15
46	.10048	.99494	.11783	.99303	.13514	.99083	.15241	.98832	.16964	.98551	14
47	.10077	.99491	.11812	.99300	.13543	.99079	.15270	.98827	.16992	.98546	13
48	.10106	.99488	.11840	.99297	.13572	.99075	.15299	.98823	.17021	.98541	12
49	.10135	.99485	.11869	.99293	.13600	.99071	.15327	.98818	.17050	.98536	11
50	.10164	.99482	.11898	.99290	.13629	.99067	.15356	.98814	.17078	.98531	10
51	.10192	.99479	.11927	.99286	.13658	.99063	.15385	.98809	.17107	.98526	9
52	.10221	.99476	.11956	.99283	.13687	.99059	.15414	.98805	.17136	.98521	8
53	.10250	.99473	.11985	.99279	.13716	.99055	.15442	.98800	.17164	.98516	7
54	.10279	.99470	.12014	.99276	.13744	.99051	.15471	.98796	.17193	.98511	6
55	.10308	.99467	.12043	.99272	.13773	.99047	.15500	.98791	.17222	.98506	5
56	.10337	.99464	.12071	.99269	.13802	.99043	.15529	.98787	.17250	.98501	4
57	.10366	.99461	.12100	.99265	.13831	.99039	.15557	.98782	.17279	.98496	3
58	.10395	.99458	.12129	.99262	.13860	.99035	.15586	.98778	.17308	.98491	2
59	.10424	.99455	.12158	.99258	.13889	.99031	.15615	.98773	.17336	.98486	1
60	.10453	.99452	.12187	.99255	.13917	.99027	.15643	.98769	.17365	.98481	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	84°		83°		82°		81°		80°		

′	10°		11°		12°		13°		14°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.17365	.98481	.19081	.98163	.20791	.97815	.22495	.97437	.24192	.97030	60
1	.17393	.98476	.19109	.98157	.20820	.97809	.22523	.97430	.24220	.97023	59
2	.17422	.98471	.19138	.98152	.20848	.97803	.22552	.97424	.24249	.97015	58
3	.17451	.98466	.19167	.98146	.20877	.97797	.22580	.97417	.24277	.97008	57
4	.17479	.98461	.19195	.98140	.20905	.97791	.22608	.97411	.24305	.97001	56
5	.17508	.98455	.19224	.98135	.20933	.97784	.22637	.97404	.24333	.96994	55
6	.17537	.98450	.19252	.98129	.20962	.97778	.22665	.97398	.24362	.96987	54
7	.17565	.98445	.19281	.98124	.20990	.97772	.22693	.97391	.24390	.96980	53
8	.17594	.98440	.19309	.98118	.21019	.97766	.22722	.97384	.24418	.96973	52
9	.17623	.98435	.19338	.98112	.21047	.97760	.22750	.97378	.24446	.96966	51
10	.17651	.98430	.19366	.98107	.21076	.97754	.22778	.97371	.24474	.96959	50
11	.17680	.98425	.19395	.98101	.21104	.97748	.22807	.97365	.24503	.96952	49
12	.17708	.98420	.19423	.98096	.21132	.97742	.22835	.97358	.24531	.96945	48
13	.17737	.98414	.19452	.98090	.21161	.97735	.22863	.97351	.24559	.96937	47
14	.17766	.98409	.19481	.98084	.21189	.97729	.22892	.97345	.24587	.96930	46
15	.17794	.98404	.19509	.98079	.21218	.97723	.22920	.97338	.24615	.96923	45
16	.17823	.98399	.19538	.98073	.21246	.97717	.22948	.97331	.24644	.96916	44
17	.17852	.98394	.19566	.98067	.21275	.97711	.22977	.97325	.24672	.96909	43
18	.17880	.98389	.19595	.98061	.21303	.97705	.23005	.97318	.24700	.96902	42
19	.17909	.98383	.19623	.98056	.21331	.97698	.23033	.97311	.24728	.96894	41
20	.17937	.98378	.19652	.98050	.21360	.97692	.23062	.97304	.24756	.96887	40
21	.17966	.98373	.19680	.98044	.21388	.97686	.23090	.97298	.24784	.96880	39
22	.17995	.98368	.19709	.98039	.21417	.97680	.23118	.97291	.24813	.96873	38
23	.18023	.98362	.19737	.98033	.21445	.97673	.23146	.97284	.24841	.96866	37
24	.18052	.98357	.19766	.98027	.21474	.97667	.23175	.97278	.24869	.96858	36
25	.18081	.98352	.19794	.98021	.21502	.97661	.23203	.97271	.24897	.96851	35
26	.18109	.98347	.19823	.98016	.21530	.97655	.23231	.97264	.24925	.96844	34
27	.18138	.98341	.19851	.98010	.21559	.97648	.23260	.97257	.24954	.96837	33
28	.18166	.98336	.19880	.98004	.21587	.97642	.23288	.97251	.24982	.96829	32
29	.18195	.98331	.19908	.97998	.21616	.97636	.23316	.97244	.25010	.96822	31
30	.18224	.98325	.19937	.97992	.21644	.97630	.23345	.97237	.25038	.96815	30
31	.18252	.98320	.19965	.97987	.21672	.97623	.23373	.97230	.25066	.96807	29
32	.18281	.98315	.19994	.97981	.21701	.97617	.23401	.97223	.25094	.96800	28
33	.18309	.98310	.20022	.97975	.21729	.97611	.23429	.97217	.25122	.96793	27
34	.18338	.98304	.20051	.97969	.21758	.97604	.23458	.97210	.25151	.96786	26
35	.18367	.98299	.20079	.97963	.21786	.97598	.23486	.97203	.25179	.96778	25
36	.18395	.98294	.20108	.97958	.21814	.97592	.23514	.97196	.25207	.96771	24
37	.18424	.98288	.20136	.97952	.21843	.97585	.23542	.97189	.25235	.96764	23
38	.18452	.98283	.20165	.97946	.21871	.97579	.23571	.97182	.25263	.96756	22
39	.18481	.98277	.20193	.97940	.21899	.97573	.23599	.97176	.25291	.96749	21
40	.18509	.98272	.20222	.97934	.21928	.97566	.23627	.97169	.25320	.96742	20
41	.18538	.98267	.20250	.97928	.21956	.97560	.23656	.97162	.25348	.96734	19
42	.18567	.98261	.20279	.97922	.21985	.97553	.23684	.97155	.25376	.96727	18
43	.18595	.98256	.20307	.97916	.22013	.97547	.23712	.97148	.25404	.96719	17
44	.18624	.98250	.20336	.97910	.22041	.97541	.23740	.97141	.25432	.96712	16
45	.18652	.98245	.20364	.97905	.22070	.97534	.23769	.97134	.25460	.96705	15
46	.18681	.98240	.20393	.97899	.22098	.97528	.23797	.97127	.25488	.96697	14
47	.18710	.98234	.20421	.97893	.22126	.97521	.23825	.97120	.25516	.96690	13
48	.18738	.98229	.20450	.97887	.22155	.97515	.23853	.97113	.25545	.96682	12
49	.18767	.98223	.20478	.97881	.22183	.97508	.23882	.97106	.25573	.96675	11
50	.18795	.98218	.20507	.97875	.22212	.97502	.23910	.97100	.25601	.96667	10
51	.18824	.98212	.20535	.97869	.22240	.97496	.23938	.97093	.25629	.96660	9
52	.18852	.98207	.20563	.97863	.22268	.97489	.23966	.97086	.25657	.96653	8
53	.18881	.98201	.20592	.97857	.22297	.97483	.23995	.97079	.25685	.96645	7
54	.18910	.98196	.20620	.97851	.22325	.97476	.24023	.97072	.25713	.96638	6
55	.18938	.98190	.20649	.97845	.22353	.97470	.24051	.97065	.25741	.96630	5
56	.18967	.98185	.20677	.97839	.22382	.97463	.24079	.97058	.25769	.96623	4
57	.18995	.98179	.20706	.97833	.22410	.97457	.24108	.97051	.25798	.96615	3
58	.19024	.98174	.20734	.97827	.22438	.97450	.24136	.97044	.25826	.96608	2
59	.19052	.98168	.20763	.97821	.22467	.97444	.24164	.97037	.25854	.96600	1
60	.19081	.98163	.20791	.97815	.22495	.97437	.24192	.97030	.25882	.96593	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	79°		78°		77°		76°		75°		

′	15°		16°		17°		18°		19°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.25882	.96593	.27564	.96126	.29237	.95630	.30902	.95106	.32557	.94552	60
1	.25910	.96585	.27592	.96118	.29265	.95622	.30929	.95097	.32584	.94542	59
2	.25938	.96578	.27620	.96110	.29293	.95613	.30957	.95088	.32612	.94533	58
3	.25966	.96570	.27648	.96102	.29321	.95605	.30985	.95079	.32639	.94523	57
4	.25994	.96562	.27676	.96094	.29348	.95596	.31012	.95070	.32667	.94514	56
5	.26022	.96555	.27704	.96086	.29376	.95588	.31040	.95061	.32694	.94504	55
6	.26050	.96547	.27731	.96078	.29404	.95579	.31068	.95052	.32722	.94495	54
7	.26079	.96540	.27759	.96070	.29432	.95571	.31095	.95043	.32749	.94485	53
8	.26107	.96532	.27787	.96062	.29460	.95562	.31123	.95033	.32777	.94476	52
9	.26135	.96524	.27815	.96054	.29487	.95554	.31151	.95024	.32804	.94466	51
10	.26163	.96517	.27843	.96046	.29515	.95545	.31178	.95015	.32832	.94457	50
11	.26191	.96509	.27871	.96037	.29543	.95536	.31206	.95006	.32859	.94447	49
12	.26219	.96502	.27899	.96029	.29571	.95528	.31233	.94997	.32887	.94438	48
13	.26247	.96494	.27927	.96021	.29599	.95519	.31261	.94988	.32914	.94428	47
14	.26275	.96486	.27955	.96013	.29626	.95511	.31289	.94979	.32942	.94418	46
15	.26303	.96479	.27983	.96005	.29654	.95502	.31316	.94970	.32969	.94409	45
16	.26331	.96471	.28011	.95997	.29682	.95493	.31344	.94961	.32997	.94399	44
17	.26359	.96463	.28039	.95989	.29710	.95485	.31372	.94952	.33024	.94390	43
18	.26387	.96456	.28067	.95981	.29737	.95476	.31399	.94943	.33051	.94380	42
19	.26415	.96448	.28095	.95972	.29765	.95467	.31427	.94933	.33079	.94370	41
20	.26443	.96440	.28123	.95964	.29793	.95459	.31454	.94924	.33106	.94361	40
21	.26471	.96433	.28150	.95956	.29821	.95450	.31482	.94915	.33134	.94351	39
22	.26500	.96425	.28178	.95948	.29849	.95441	.31510	.94906	.33161	.94342	38
23	.26528	.96417	.28206	.95940	.29876	.95433	.31537	.94897	.33189	.94332	37
24	.26556	.96410	.28234	.95931	.29904	.95424	.31565	.94888	.33216	.94322	36
25	.26584	.96402	.28262	.95923	.29932	.95415	.31593	.94878	.33244	.94313	35
26	.26612	.96394	.28290	.95915	.29960	.95407	.31620	.94869	.33271	.94303	34
37	.26640	.96386	.28318	.95907	.29987	.95398	.31648	.94860	.33298	.94293	33
28	.26668	.96379	.28346	.95898	.30015	.95389	.31675	.94851	.33326	.94284	32
29	.26696	.96371	.28374	.95890	.30043	.95380	.31703	.94842	.33353	.94274	31
30	.26724	.96363	.28402	.95882	.30071	.95372	.31730	.94832	.33381	.94264	30
31	.26752	.96355	.28429	.95874	.30098	.95363	.31758	.94823	.33408	.94254	29
32	.26780	.96347	.28457	.95865	.30126	.95354	.31786	.94814	.33436	.94245	28
33	.26808	.96340	.28485	.95857	.30154	.95345	.31813	.94805	.33463	.94235	27
34	.26836	.96332	.28513	.95849	.30182	.95337	.31841	.94795	.33490	.94225	26
35	.26864	.96324	.28541	.95841	.30209	.95328	.31868	.94786	.33518	.94215	25
36	.26892	.96316	.28569	.95832	.30237	.95319	.31896	.94777	.33545	.94206	24
37	.26920	.96308	.28597	.95824	.30265	.95310	.31923	.94768	.33573	.94196	23
38	.26948	.96301	.28625	.95816	.30292	.95301	.31951	.94758	.33600	.94186	22
39	.26976	.96293	.28652	.95807	.30320	.95293	.31979	.94749	.33627	.94176	21
40	.27004	.96285	.28680	.95799	.30348	.95284	.32006	.94740	.33655	.94167	20
41	.27032	.96277	.28708	.95791	.30376	.95275	.32034	.94730	.33682	.94157	19
42	.27060	.96269	.28736	.95782	.30403	.95266	.32061	.94721	.33710	.94147	18
43	.27088	.96261	.28764	.95774	.30431	.95257	.32089	.94712	.23737	.94137	17
44	.27116	.96253	.28792	.95766	.30459	.95248	.32116	.94702	.33764	.94127	16
45	.27144	.96246	.28820	.95757	.30486	.95240	.32144	.94693	.33792	.94118	15
46	.27172	.96238	.28847	.95749	.30514	.95231	.32171	.94684	.33819	.94108	14
47	.27200	.96230	.28875	.95740	.30542	.95222	.32199	.94674	.33846	.94098	13
48	.27228	.96222	.28903	.95732	.30570	.95213	.32227	.94665	.33874	.94088	12
49	.27256	.96214	.28931	.95724	.30597	.95204	.32254	.94656	.33901	.94078	11
50	.27284	.96206	.28959	.95715	.30625	.95195	.32282	.94646	.33929	.94068	10
51	.27312	.96198	.28987	.95707	.30653	.95186	.32309	.94637	.33956	.94058	9
52	.27340	.96190	.29015	.95698	.30680	.95177	.32337	.94627	.33983	.94049	8
53	.27368	.96182	.29042	.95690	.30708	.95168	.32364	.94618	.34011	.94039	7
54	.27396	.96174	.29070	.95681	.30736	.95159	.32392	.94609	.34038	.94029	6
55	.27424	.96166	.29098	.95673	.30763	.95150	.32419	.94599	.34065	.94019	5
56	.27452	.96158	.29126	.95664	.30791	.95142	.32447	.94590	.34093	.94009	4
57	.27480	.96150	.29154	.95656	.30819	.95133	.32474	.94580	.34120	.93999	3
58	.27508	.96142	.29182	.95647	.30846	.95124	.32502	.94571	.34147	.93989	2
59	.27536	.96134	.29209	.95639	.30874	.95115	.32529	.94561	.34175	.93979	1
60	.27564	.96126	.29237	.95630	.30902	.95106	.32557	.94552	.34202	.93969	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	74°		73°		72°		71°		70°		

′	20°		21°		22°		23°		24°		
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.34202	.93969	.35837	.93358	.37461	.92718	.39073	.92050	.40674	.91355	60
1	.34229	.93959	.35864	.93348	.37488	.92707	.39100	.92039	.40700	.91343	59
2	.34257	.93949	.35891	.93337	.37515	.92697	.39127	.92028	.40727	.91331	58
3	.34284	.93939	.35918	.93327	.37542	.92686	.39153	.92016	.40753	.91319	57
4	.34311	.93929	.35945	.93316	.37569	.92675	.39180	.92005	.40780	.91307	56
5	.34339	.93919	.35973	.93306	.37595	.92664	.39207	.91994	.40806	.91295	55
6	.34366	.93909	.36000	.93295	.37622	.92653	.39234	.91982	.40833	.91283	54
7	.34393	.93899	.36027	.93285	.37649	.92642	.39260	.91971	.40860	.91272	53
8	.34421	.93889	.36054	.93274	.37676	.92631	.39287	.91959	.40886	.91260	52
9	.34448	.93879	.36081	.93264	.37703	.92620	.39314	.91948	.40913	.91248	51
10	.34475	.93869	.36108	.93253	.37730	.92609	.39341	.91936	.40939	.91236	50
11	.34503	.93859	.36135	.93243	.37757	.92598	.39367	.91925	.40966	.91224	49
12	.34530	.93849	.36162	.93232	.37784	.92587	.39394	.91914	.40992	.91212	48
13	.34557	.93839	.36190	.93222	.37811	.92576	.39421	.91902	.41019	.91200	47
14	.34584	.93829	.36217	.93211	.37838	.92565	.39448	.91891	.41045	.91188	46
15	.34612	.93819	.36244	.93201	.37865	.92554	.39474	.91879	.41072	.91176	45
16	.34639	.93809	.36271	.93190	.37892	.92543	.39501	.91868	.41098	.91164	44
17	.34666	.93799	.36298	.93180	.37919	.92532	.39528	.91856	.41125	.91152	43
18	.34694	.93789	.36325	.93169	.37946	.92521	.39555	.91845	.41151	.91140	42
19	.34721	.93779	.36352	.93159	.37973	.92510	.39581	.91833	.41178	.91128	41
20	.34748	.93769	.36379	.93148	.37999	.92499	.39608	.91822	.41204	.91116	40
21	.34775	.93759	.36406	.93137	.38026	.92488	.39635	.91810	.41231	.91104	39
22	.34803	.93748	.36434	.93127	.38053	.92477	.39661	.91799	.41257	.91092	38
23	.34830	.93738	.36461	.93116	.38080	.92466	.39688	.91787	.41284	.91080	37
24	.34857	.93728	.36488	.93106	.38107	.92455	.39715	.91775	.41310	.91068	36
25	.34884	.93718	.36515	.93095	.38134	.92444	.39741	.91764	.41337	.91056	35
26	.34912	.93708	.36542	.93084	.38161	.92432	.39768	.91752	.41363	.91044	34
27	.34939	.93698	.36569	.93074	.38188	.92421	.39795	.91741	.41390	.91032	33
28	.34966	.93688	.36596	.93063	.38215	.92410	.39822	.91729	.41416	.91020	32
29	.34993	.93677	.36623	.93052	.28241	.92399	.39848	.91718	.41443	.91008	31
30	.35021	.93667	.36650	.93042	.38268	.92388	.39875	.91706	.41469	.90996	30
31	.35048	.93657	.36677	.93031	.38295	.92377	.39902	.91694	.41496	.90984	29
32	.35075	.93647	.36704	.93020	.38322	.92366	.39928	.91683	.41522	.90972	28
33	.35102	.93637	.36731	.93010	.38349	.92355	.39955	.91671	.41549	.90960	27
34	.35130	.93626	.36758	.92999	.38376	.92343	.39982	.91660	.41575	.90948	26
35	.35157	.93616	.36785	.92988	.38403	.92332	.40008	.91648	.41602	.90936	25
36	.35184	.93606	.36812	.92978	.38430	.92321	.40035	.91636	.41628	.90924	24
37	.35211	.93596	.36839	.92967	.38456	.92310	.40062	.91625	.41655	.90911	23
38	.35239	.93585	.36867	.92956	.38483	.92299	.40088	.91613	.41681	.90899	22
39	.35266	.93575	.36894	.92945	.38510	.92287	.40115	.91601	.41707	.90887	21
40	.35293	.93565	.36921	.92935	.38537	.92276	.40141	.91590	.41734	.90875	20
41	.35320	.93555	.36948	.92924	.38564	.92265	.40168	.91578	.41760	.90863	19
42	.35347	.93544	.36975	.92913	.38591	.92254	.40195	.91566	.41787	.90851	18
43	.35375	.93534	.37002	.92902	.38617	.92243	.40221	.91555	.41813	.90839	17
44	.35402	.93524	.37029	.92892	.38644	.92231	.40248	.91543	.41840	.90826	16
45	.35429	.93514	.37056	.92881	.38671	.92220	.40275	.91531	.41866	.90814	15
46	.35456	.93503	.37083	.92870	.38698	.92209	.40301	.91519	.41892	.90802	14
47	.35484	.93493	.37110	.92859	.38725	.92198	.40328	.91508	.41919	.90790	13
48	.35511	.93483	.37137	.92849	.38752	.92186	.40355	.91496	.41945	.90778	12
49	.35538	.93472	.37164	.92838	.38778	.92175	.40381	.91484	.41972	.90766	11
50	.35565	.93462	.37191	.92827	.38805	.92164	.40408	.91472	.41998	.90753	10
51	.35592	.93452	.37218	.92816	.38832	.92152	.40434	.91461	.42024	.90741	9
52	.35619	.93441	.37245	.92805	.38859	.92141	.40461	.91449	.42051	.90729	8
53	.35647	.93431	.37272	.92794	.38886	.92130	.40488	.91437	.42077	.90717	7
54	.35674	.93420	.37299	.92784	.38912	.92119	.40514	.91425	.42104	.90704	6
55	.35701	.93410	.37326	.92773	.38939	.92107	.40541	.91414	.42130	.90692	5
56	.35728	.93400	.37353	.92762	.38966	.92096	.40567	.91402	.42156	.90680	4
57	.35755	.93389	.37380	.92751	.38993	.92085	.40594	.91390	.42183	.90668	3
58	.35782	.93379	.37407	.92740	.39020	.92073	.40621	.91378	.42209	.90655	2
59	.35810	.93368	.37434	.92729	.39046	.92062	.40647	.91366	.42235	.90643	1
60	.35837	.93358	.37461	.92718	.39073	.92050	.40674	.91355	.42262	.90631	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	69°		68°		67°		66°		65°		

′	25°		26°		27°		28°		29°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.42262	.90631	.43837	.89879	.45399	.89101	.46947	.88295	.48481	.87462	60
1	.42288	.90618	.43863	.89867	.45425	.89087	.46973	.88281	.48506	.87448	59
2	.42315	.90606	.43889	.89854	.45451	.89074	.46999	.88267	.48532	.87434	58
3	.42341	.90594	.43916	.89841	.45477	.89061	.47024	.88254	.48557	.87420	57
4	.42367	.90582	.43942	.89828	.45503	.89048	.47050	.88240	.48583	.87406	56
5	.42394	.90569	.43968	.89816	.45529	.89035	.47076	.88226	.48608	.87391	55
6	.42420	.90557	.43994	.89803	.45554	.89021	.47101	.88213	.48634	.87377	54
7	.42446	.90545	.44020	.89790	.45580	.89008	.47127	.88199	.48659	.87363	53
8	.42473	.90532	.44046	.89777	.45606	.88995	.47153	.88185	.48684	.87349	52
9	.42499	.90520	.44072	.89764	.45632	.88981	.47178	.88172	.48710	.87335	51
10	.42525	.90507	.44098	.89752	.45658	.88968	.47204	.88158	.48735	.87321	50
11	.42552	.90495	.44124	.89739	.45684	.88955	.47229	.88144	.48761	.87306	49
12	.42578	.90483	.44151	.89726	.45710	.88942	.47255	.88130	.48786	.87292	48
13	.42604	.90470	.44177	.89713	.45736	.88928	.47281	.88117	.48811	.87278	47
14	.42631	.90458	.44203	.89700	.45762	.88915	.47306	.88103	.48837	.87264	46
15	.42657	.90446	.44229	.89687	.45787	.88902	.47332	.88089	.48862	.87250	45
16	.42683	.90433	.44255	.89674	.45813	.88888	.47358	.88075	.48888	.87235	44
17	.42709	.90421	.44281	.89662	.45839	.88875	.47383	.88062	.48913	.87221	43
18	.42736	.90408	.44307	.89649	.45865	.88862	.47409	.88048	.48938	.87207	42
19	.42762	.90396	.44333	.89636	.45891	.88848	.47434	.88034	.48964	.87193	41
20	.42788	.90383	.44359	.89623	.45917	.88835	.47460	.88020	.48989	.87178	40
21	.42815	.90371	.44385	.89610	.45942	.88822	.47486	.88006	.49014	.87164	39
22	.42841	.90358	.44411	.89597	.45968	.88808	.47511	.87993	.49040	.87150	38
23	.42867	.90346	.44437	.89584	.45994	.88795	.47537	.87979	.49065	.87136	37
24	.42894	.90334	.44464	.89571	.46020	.88782	.47562	.87965	.49090	.87121	36
25	.42920	.90321	.44490	.89558	.46046	.88768	.47588	.87951	.49116	.87107	35
26	.42946	.90309	.44516	.89545	.46072	.88755	.47614	.87937	.49141	.87093	34
27	.42972	.90296	.44542	.89532	.46097	.88741	.47639	.87923	.49166	.87079	33
28	.42999	.90284	.44568	.89519	.46123	.88728	.47665	.87909	.49192	.87064	32
29	.43025	.90271	.44594	.89506	.46149	.88715	.47690	.87896	.49217	.87050	31
30	.43051	.90259	.44620	.89493	.46175	.88701	.47716	.87882	.49242	.87036	30
31	.43077	.90246	.44646	.89480	.46201	.88688	.47741	.87868	.49268	.87021	29
32	.43104	.90233	.44672	.89467	.46226	.88674	.47767	.87854	.49293	.87007	28
33	.43130	.90221	.44698	.89454	.46252	.88661	.47793	.87840	.49318	.86993	27
34	.43156	.90208	.44724	.89441	.46278	.88647	.47818	.87826	.49344	.86978	26
35	.43182	.90196	.44750	.89428	.46304	.88634	.47844	.87812	.49369	.86964	25
36	.43209	.90183	.44776	.89415	.46330	.88620	.47869	.87798	.49394	.86949	24
37	.43235	.90171	.44802	.89402	.46355	.88607	.47895	.87784	.49419	.86935	23
38	.43261	.90158	.44828	.89389	.46381	.88593	.47920	.87770	.49445	.86921	22
39	.43287	.90146	.44854	.89376	.46407	.88580	.47946	.87756	.49470	.86906	21
40	.43313	.90133	.44880	.89363	.46433	.88566	.47971	.87743	.49495	.86892	20
41	.43340	.90120	.44906	.89350	.46458	.88553	.47997	.87729	.49521	.86878	19
42	.43366	.90108	.44932	.89337	.46484	.88539	.48022	.87715	.49546	.86863	18
43	.43392	.90095	.44958	.89324	.46510	.88526	.48048	.87701	.49571	.86849	17
44	.43418	.90082	.44984	.89311	.46536	.88512	.48073	.87687	.49596	.86834	16
45	.43445	.90070	.45010	.89298	.46561	.88499	.48099	.87673	.49622	.86820	15
46	.43471	.90057	.45036	.89285	.46587	.88485	.48124	.87659	.49647	.86805	14
47	.43497	.90045	.45062	.89272	.46613	.88472	.48150	.87645	.49672	.86791	13
48	.43523	.90032	.45088	.89259	.46639	.88458	.48175	.87631	.49697	.86777	12
49	.43549	.90019	.45114	.89245	.46664	.88445	.48201	.87617	.49723	.86762	11
50	.43575	.90007	.45140	.89232	.46690	.88431	.48226	.87603	.49748	.86748	10
51	.43602	.89994	.45166	.89219	.46716	.88417	.48252	.87589	.49773	.86733	9
52	.43628	.89981	.45192	.89206	.46742	.88404	.48277	.87575	.49798	.86719	8
53	.43654	.89968	.45218	.89193	.46767	.88390	.48303	.87561	.49824	.86704	7
54	.43680	.89956	.45243	.89180	.46793	.88377	.48328	.87546	.49849	.86690	6
55	.43706	.89943	.45269	.89167	.46819	.88363	.48354	.87532	.49874	.86675	5
56	.43733	.89930	.45295	.89153	.46844	.88349	.48379	.87518	.49899	.86661	4
57	.43759	.89918	.45321	.89140	.46870	.88336	.48405	.87504	.49924	.86646	3
58	.43785	.89905	.45347	.89127	.46896	.88322	.48430	.87490	.49950	.86632	2
59	.43811	.89892	.45373	.89114	.46921	.88308	.48456	.87476	.49975	.86617	1
60	.43837	.89879	.45399	.89101	.46947	.88295	.48481	.87462	.50000	.86603	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	64°		63°		62°		61°		60°		

′	30° Sine	30° Cosin	31° Sine	31° Cosin	32° Sine	32° Cosin	33° Sine	33° Cosin	34° Sine	34° Cosin	′
0	.50000	.86603	.51504	.85717	.52992	.84805	.54464	.83867	.55919	.82904	60
1	.50025	.86588	.51529	.85702	.53017	.84789	.54488	.83851	.55943	.82887	59
2	.50050	.86573	.51554	.85687	.53041	.84774	.54513	.83835	.55968	.82871	58
3	.50076	.86559	.51579	.85672	.53066	.84759	.54537	.83819	.55992	.82855	57
4	.50101	.86544	.51604	.85657	.53091	.84743	.54561	.83804	.56016	.82839	56
5	.50126	.86530	.51628	.85642	.53115	.84728	.54586	.83788	.56040	.82822	55
6	.50151	.86515	.51653	.85627	.53140	.84712	.54610	.83772	.56064	.82806	54
7	.50176	.86501	.51678	.85612	.53164	.84697	.54635	.83756	.56088	.82790	53
8	.50201	.86486	.51703	.85597	.53189	.84681	.54659	.83740	.56112	.82773	52
9	.50227	.86471	.51728	.85582	.53214	.84666	.54683	.83724	.56136	.82757	51
10	.50252	.86457	.51753	.85567	.53238	.84650	.54708	.83708	.56160	.82741	50
11	.50277	.86442	.51778	.85551	.53263	.84635	.54732	.83692	.56184	.82724	49
12	.50302	.86427	.51803	.85536	.53288	.84619	.54756	.83676	.56208	.82708	48
13	.50327	.86413	.51828	.85521	.53312	.84604	.54781	.83660	.56232	.82692	47
14	.50352	.86398	.51852	.85506	.53337	.84588	.54805	.83645	.56256	.82675	46
15	.50377	.86384	.51877	.85491	.53361	.84573	.54829	.83629	.56280	.82659	45
16	.50403	.86369	.51902	.85476	.53386	.84557	.54854	.83613	.56305	.82643	44
17	.50428	.86354	.51927	.85461	.53411	.84542	.54878	.83597	.56329	.82626	43
18	.50453	.86340	.51952	.85446	.53435	.84526	.54902	.83581	.56353	.82610	42
19	.50478	.86325	.51977	.85431	.53460	.84511	.54927	.83565	.56377	.82593	41
20	.50503	.86310	.52002	.85416	.53484	.84495	.54951	.83549	.56401	.82577	40
21	.50528	.86295	.52026	.85401	.53509	.84480	.54975	.83533	.56425	.82561	39
22	.50553	.86281	.52051	.85385	.53534	.84464	.54999	.83517	.56449	.82544	38
23	.50578	.86266	.52076	.85370	.53558	.84448	.55024	.83501	.56473	.82528	37
24	.50603	.86251	.52101	.85355	.53583	.84433	.55048	.83485	.56497	.82511	36
25	.50628	.86237	.52126	.85340	.53607	.84417	.55072	.83469	.56521	.82495	35
26	.50654	.86222	.52151	.85325	.53632	.84402	.55097	.83453	.56545	.82478	34
27	.50679	.86207	.52175	.85310	.53656	.84386	.55121	.83437	.56569	.82462	33
28	.50704	.86192	.52200	.85294	.53681	.84370	.55145	.83421	.56593	.82446	32
29	.50729	.86178	.52225	.85279	.53705	.84355	.55169	.83405	.56617	.82429	31
30	.50754	.86163	.52250	.85264	.53730	.84339	.55194	.83389	.56641	.82413	30
31	.50779	.86148	.52275	.85249	.53754	.84324	.55218	.83373	.56665	.82396	29
32	.50804	.86133	.52299	.85234	.53779	.84308	.55242	.83356	.56689	.82380	28
33	.50829	.86119	.52324	.85218	.53804	.84292	.55266	.83340	.56713	.82363	27
34	.50854	.86104	.52349	.85203	.53828	.84277	.55291	.83324	.56736	.82347	26
35	.50879	.86089	.52374	.85188	.53853	.84261	.55315	.83308	.56760	.82330	25
36	.50904	.86074	.52399	.85173	.53877	.84245	.55339	.83292	.56784	.82314	24
37	.50929	.86059	.52423	.85157	.53902	.84230	.55363	.83276	.56808	.82297	23
38	.50954	.86045	.52448	.85142	.53926	.84214	.55388	.83260	.56832	.82281	22
39	.50979	.86030	.52473	.85127	.53951	.84198	.55412	.83244	.56856	.82264	21
40	.51004	.86015	.52498	.85112	.53975	.84182	.55436	.83228	.56880	.82248	20
41	.51029	.86000	.52522	.85096	.54000	.84167	.55460	.83212	.56904	.82231	19
42	.51054	.85985	.52547	.85081	.54024	.84151	.55484	.83195	.56928	.82214	18
43	.51079	.85970	.52572	.85066	.54049	.84135	.55509	.83179	.56952	.82198	17
44	.51104	.85956	.52597	.85051	.54073	.84120	.55533	.83163	.56976	.82181	16
45	.51129	.85941	.52621	.85035	.54097	.84104	.55557	.83147	.57000	.82165	15
46	.51154	.85926	.52646	.85020	.54122	.84088	.55581	.83131	.57024	.82148	14
47	.51179	.85911	.52671	.85005	.54146	.84072	.55605	.83115	.57047	.82132	13
48	.51204	.85896	.52696	.84989	.54171	.84057	.55630	.83098	.57071	.82115	12
49	.51229	.85881	.52720	.84974	.54195	.84041	.55654	.83082	.57095	.82098	11
50	.51254	.85866	.52745	.84959	.54220	.84025	.55678	.83066	.57119	.82082	10
51	.51279	.85851	.52770	.84943	.54244	.84009	.55702	.83050	.57143	.82065	9
52	.51304	.85836	.52794	.84928	.54269	.83994	.55726	.83034	.57167	.82048	8
53	.51329	.85821	.52819	.84913	.54293	.83978	.55750	.83017	.57191	.82032	7
54	.51354	.85806	.52844	.84897	.54317	.83962	.55775	.83001	.57215	.82015	6
55	.51379	.85792	.52869	.84882	.54342	.83946	.55799	.82985	.57238	.81999	5
56	.51404	.85777	.52893	.84866	.54366	.83930	.55823	.82969	.57262	.81982	4
57	.51429	.85762	.52918	.84851	.54391	.83915	.55847	.82953	.57286	.81965	3
58	.51454	.85747	.52943	.84836	.54415	.83899	.55871	.82936	.57310	.81949	2
59	.51479	.85732	.52967	.84820	.54440	.83883	.55895	.82920	.57334	.81932	1
60	.51504	.85717	.52992	.84805	.54464	.83867	.55919	.82904	.57358	.81915	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	59°		58°		57°		56°		55°		

′	35°		36°		37°		38°		39°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.57358	.81915	.58779	.80902	.60182	.79864	.61566	.78801	.62932	.77715	60
1	.57381	.81899	.58802	.80885	.60205	.79846	.61589	.78783	.62955	.77696	59
2	.57405	.81882	.58826	.80867	.60228	.79829	.61612	.78765	.62977	.77678	58
3	.57429	.81865	.58849	.80850	.60251	.79811	.61635	.78747	.63000	.77660	57
4	.57453	.81848	.58873	.80833	.60274	.79793	.61658	.78729	.63022	.77641	56
5	.57477	.81832	.58896	.80816	.60298	.79776	.61681	.78711	.63045	.77623	55
6	.57501	.81815	.58920	.80799	.60321	.79758	.61704	.78694	.63068	.77605	54
7	.57524	.81798	.58943	.80782	.60344	.79741	.61726	.78676	.63090	.77586	53
8	.57548	.81782	.58967	.80765	.60367	.79723	.61749	.78658	.63113	.77568	52
9	.57572	.81765	.58990	.80748	.60390	.79706	.61772	.78640	.63135	.77550	51
10	.57596	.81748	.59014	.80730	.60414	.79688	.61795	.78622	.63158	.77531	50
11	.57619	.81731	.59037	.80713	.60437	.79671	.61818	.78604	.63180	.77513	40
12	.57643	.81714	.59061	.80696	.60460	.79653	.61841	.78586	.63203	.77494	48
13	.57667	.81698	.59084	.80679	.60483	.79635	.61864	.78568	.63225	.77476	47
14	.57691	.81681	.59108	.80662	.60506	.79618	.61887	.78550	.63248	.77458	46
15	.57715	.81664	.59131	.80644	.60529	.79600	.61909	.78532	.63271	.77439	45
16	.57738	.81647	.59154	.80627	.60553	.79583	.61932	.78514	.63293	.77421	44
17	.57762	.81631	.59178	.80610	.60576	.79565	.61955	.78496	.63316	.77402	43
18	.57786	.81614	.59201	.80593	.60599	.79547	.61978	.78478	.63338	.77384	42
19	.57810	.81597	.59225	.80576	.60622	.79530	.62001	.78460	.63361	.77366	41
20	.57833	.81580	.59248	.80558	.60645	.79512	.62024	.78442	.63383	.77347	40
21	.57857	.81563	.59272	.80541	.60668	.79494	.62046	.78424	.63406	.77329	39
22	.57881	.81546	.59295	.80524	.60691	.79477	.62069	.78405	.63428	.77310	38
23	.57904	.81530	.59318	.80507	.60714	.79459	.62092	.78387	.63451	.77292	37
24	.57928	.81513	.59342	.80489	.60738	.79441	.62115	.78369	.63473	.77273	36
25	.57952	.81496	.59365	.80472	.60761	.79424	.62138	.78351	.63496	.77255	35
26	.57976	.81479	.59389	.80455	.60784	.79406	.62160	.78333	.63518	.77236	34
27	.57999	.81462	.59412	.80438	.60807	.79388	.62183	.78315	.63540	.77218	33
28	.58023	.81445	.59436	.80420	.60830	.79371	.62206	.78297	.63563	.77199	32
29	.58047	.81428	.59459	.80403	.60853	.79353	.62229	.78279	.63585	.77181	31
30	.58070	.81412	.59482	.80386	.60876	.79335	.62251	.78261	.63608	.77162	30
31	.58094	.81395	.59506	.80368	.60899	.79318	.62274	.78243	.63630	.77144	29
32	.58118	.81378	.59529	.80351	.60922	.79300	.62297	.78225	.63653	.77125	28
33	.58141	.81361	.59552	.80334	.60945	.79282	.62320	.78206	.63675	.77107	27
34	.58165	.81344	.59576	.80316	.60968	.79264	.62342	.78188	.63698	.77088	26
35	.58189	.81327	.59599	.80299	.60991	.79247	.62365	.78170	.63720	.77070	25
36	.58212	.82310	.59622	.80282	.61015	.79229	.62388	.78152	.63742	.77051	24
37	.58236	.81293	.59646	.80264	.61038	.79211	.62411	.78134	.63765	.77033	23
38	.58260	.81276	.59669	.80247	.61061	.79193	.62433	.78116	.63787	.77014	22
39	.58283	.81259	.59693	.80230	.61084	.79176	.62456	.78098	.63810	.76996	21
40	.58307	.81242	.59716	.80212	.61107	.79158	.62479	.78079	.63832	.76977	20
41	.58330	.81225	.59739	.80195	.61130	.79140	.62502	.78061	.63854	.76959	19
42	.58354	.81208	.59763	.80178	.61153	.79122	.62524	.78043	.63877	.76940	18
43	.58378	.81191	.59786	.80160	.61176	.79105	.62547	.78025	.63899	.76921	17
44	.58401	.81174	.59809	.80143	.61199	.79087	.62570	.78007	.63922	.76903	16
45	.58425	.81157	.59832	.80125	.61222	.79069	.62592	.77988	.63944	.76884	15
46	.58449	.81140	.59856	.80108	.61245	.79051	.62615	.77970	.63966	.76866	14
47	.58472	.81123	.59879	.80091	.61268	.79033	.62638	.77952	.63989	.76847	13
48	.58496	.81106	.59902	.80073	.61291	.79016	.62660	.77934	.64011	.76828	12
49	.58519	.81089	.59926	.80056	.61314	.78998	.62683	.77916	.64033	.76810	11
50	.58543	.81072	.59949	.80038	.61337	.78980	.62706	.77897	.64056	.76791	10
51	.58567	.81055	.59972	.80021	.61360	.78962	.62728	.77879	.64078	.76772	9
52	.58590	.81038	.59995	.80003	.61383	.78944	.62751	.77861	.64100	.76754	8
53	.58614	.81021	.60019	.79986	.61406	.78926	.62774	.77843	.64123	.76735	7
54	.58637	.81004	.60042	.79968	.61429	.78908	.62796	.77824	.64145	.76717	6
55	.58661	.80987	.60065	.79951	.61451	.78891	.62819	.77806	.64167	.76698	5
56	.58684	.80970	.60089	.79934	.61474	.78873	.62842	.77788	.64190	.76679	4
57	.58708	.80953	.60112	.79916	.61497	.78855	.62864	.77769	.64212	.76661	3
58	.58731	.80936	.60135	.79899	.61520	.78837	.62887	.77751	.64234	.76642	2
59	.58755	.80919	.60158	.79881	.61543	.78819	.62909	.77733	.64256	.76623	1
60	.58779	.80902	.60182	.79864	.61566	.78801	.62932	.77715	.64279	.76604	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	54°		53°		52°		51°		50°		

′	40°		41°		42°		43°		44°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.64279	.76604	.65606	.75471	.66913	.74314	.68200	.73135	.69466	.71934	60
1	.64301	.76586	.65628	.75452	.66935	.74295	.68221	.73116	.69487	.71914	59
2	.64323	.76567	.65650	.75433	.66956	.74276	.68242	.73096	.69508	.71894	58
3	.64346	.76548	.65672	.75414	.66978	.74256	.68264	.73076	.69529	.71873	57
4	.64368	.76530	.65694	.75395	.66999	.74237	.68285	.73056	.69549	.71853	56
5	.64390	.76511	.65716	.75375	.67021	.74217	.68306	.73036	.69570	.71833	55
6	.64412	.76492	.65738	.75356	.67043	.74198	.68327	.73016	.69591	.71813	54
7	.64435	.76473	.65759	.75337	.67064	.74178	.68349	.72996	.69612	.71792	53
8	.64457	.76455	.65781	.75318	.67086	.74159	.68370	.72976	.69633	.71772	52
9	.64479	.76436	.65803	.75299	.67107	.74139	.68391	.72957	.69654	.71752	51
10	.64501	.76417	.65825	.75280	.67129	.74120	.68412	.72937	.69675	.71732	50
11	.64524	.76398	.65847	.75261	.67151	.74100	.68434	.72917	.69696	.71711	49
12	.64546	.76380	.65869	.75241	.67172	.74080	.68455	.72897	.69717	.71691	48
13	.64568	.76361	.65891	.75222	.67194	.74061	.68476	.72877	.69737	.71671	47
14	.64590	.76342	.65913	.75203	.67215	.74041	.68497	.72857	.69758	.71650	46
15	.64612	.76323	.65935	.75184	.67237	.74022	.68518	.72837	.69779	.71630	45
16	.64635	.76304	.65956	.75165	.67258	.74002	.68539	.72817	.69800	.71610	44
17	.64657	.76286	.65978	.75146	.67280	.73983	.68561	.72797	.69821	.71590	43
18	.64679	.76267	.66000	.75126	.67301	.73963	.68582	.72777	.69842	.71569	42
19	.64701	.76248	.66022	.75107	.67323	.73944	.68603	.72757	.69862	.71549	41
20	.64723	.76229	.66044	.75088	.67344	.73924	.68624	.72737	.69883	.71529	40
21	.64746	.76210	.66066	.75069	.67366	.73904	.68645	.72717	.69904	.71508	39
22	.64768	.76192	.66088	.75050	.67387	.73885	.68666	.72697	.69925	.71488	38
23	.64790	.76173	.66109	.75030	.67409	.73865	.68688	.72677	.69946	.71468	37
24	.64812	.76154	.66131	.75011	.67430	.73846	.68709	.72657	.69966	.71447	36
25	.64834	.76135	.66153	.74992	.67452	.73826	.68730	.72637	.69987	.71427	35
26	.64856	.76116	.66175	.74973	.67473	.73806	.68751	.72617	.70008	.71407	34
27	.64878	.76097	.66197	.74953	.67495	.73787	.68772	.72597	.70029	.71386	33
28	.64901	.76078	.66218	.74934	.67516	.73767	.68793	.72577	.70049	.71366	32
29	.64923	.76059	.66240	.74915	.67538	.73747	.68814	.72557	.70070	.71345	31
30	.64945	.76041	.66262	.74896	.67559	.73728	.68835	.72537	.70091	.71325	30
31	.64967	.76022	.66284	.74876	.67580	.73708	.68857	.72517	.70112	.71305	29
32	.64989	.76003	.66306	.74857	.67602	.73688	.68878	.72497	.70132	.71284	28
33	.65011	.75984	.66327	.74838	.67623	.73669	.68899	.72477	.70153	.71264	27
34	.65033	.75965	.66349	.74818	.67645	.73649	.68920	.72457	.70174	.71243	26
35	.65055	.75946	.66371	.74799	.67666	.73629	.68941	.72437	.70195	.71223	25
36	.65077	.75927	.66393	.74780	.67688	.73610	.68962	.72417	.70215	.71203	24
37	.65100	.75908	.66414	.74760	.67709	.73590	.68983	.72397	.70236	.71182	23
38	.65122	.75889	.66436	.74741	.67730	.73570	.69004	.72377	.70257	.71162	22
39	.65144	.75870	.66458	.74722	.67752	.73551	.69025	.72357	.70277	.71141	21
40	.65166	.75851	.66480	.74703	.67773	.73531	.69046	.72337	.70298	.71121	20
41	.65188	.75832	.66501	.74683	.67795	.73511	.69067	.72317	.70319	.71100	19
42	.65210	.75813	.66523	.74664	.67816	.73491	.69088	.72297	.70339	.71080	18
43	.65232	.75794	.66545	.74644	.67837	.73472	.69109	.72277	.70360	.71059	17
44	.65254	.75775	.66566	.74625	.67859	.73452	.69130	.72257	.70381	.71039	16
45	.65276	.75756	.66588	.74606	.67880	.73432	.69151	.72236	.70401	.71019	15
46	.65298	.75738	.66610	.74586	.67901	.73413	.69172	.72216	.70422	.70998	14
47	.65320	.75719	.66632	.74567	.67923	.73393	.69193	.72196	.70443	.70978	13
48	.65342	.75700	.66653	.74548	.67944	.73373	.69214	.72176	.70463	.70957	12
49	.65364	.75680	.66675	.74528	.67965	.73353	.69235	.72156	.70484	.70937	11
50	.65386	.75661	.66697	.74509	.67987	.73333	.69256	.72136	.70505	.70916	10
51	.65408	.75642	.66718	.74489	.68008	.73314	.69277	.72116	.70525	.70896	9
52	.65430	.75623	.66740	.74470	.68029	.73294	.69298	.72095	.70546	.70875	8
53	.65452	.75604	.66762	.74451	.68051	.73274	.69319	.72075	.70567	.70855	7
54	.65474	.75585	.66783	.74431	.68072	.73254	.69340	.72055	.70587	.70834	6
55	.65496	.75566	.66805	.74412	.68093	.73234	.69361	.72035	.70608	.70813	5
56	.65518	.75547	.66827	.74392	.68115	.73215	.69382	.72015	.70628	.70793	4
57	.65540	.75528	.66848	.74373	.68136	.73195	.69403	.71995	.70649	.70772	3
58	.65562	.75509	.66870	.74353	.68157	.73175	.69424	.71974	.70670	.70752	2
59	.65584	.75490	.66891	.74334	.68179	.73155	.69445	.71954	.70690	.70731	1
60	.65606	.75471	.66913	74314	.68200	.73135	.69466	.71934	.70711	.70711	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	49°		48°		47°		46°		45°		

′	0° Tang	0° Cotang	1° Tang	1° Cotang	2° Tang	2° Cotang	3° Tang	3° Cotang	′
0	.00000	Infinite.	.01746	57.2900	.03492	28.6363	.05241	19.0811	60
1	.00029	3437.75	.01775	56.3506	.03521	28.3994	.05270	18.9755	59
2	.00058	1718.87	.01804	55.4415	.03550	28.1664	.05299	18.8711	58
3	.00087	1145.92	.01833	54.5613	.03579	27.9372	.05328	18.7678	57
4	.00116	859.436	.01862	53.7086	.03609	27.7117	.05357	18.6656	56
5	.00145	687.549	.01891	52.8821	.03638	27.4899	.05387	18.5645	55
6	.00175	572.957	.01920	52.0807	.03667	27.2715	.05416	18.4645	54
7	.00204	491.106	.01949	51.3032	.03696	27.0566	.05445	18.3655	53
8	.00233	429.718	.01978	50.5485	.03725	26.8450	.05474	18.2677	52
9	.00262	381.971	.02007	49.8157	.03754	26.6367	.05503	18.1708	51
10	.00291	343.774	.02036	49.1039	.03783	26.4316	.05533	18.0750	50
11	.00320	312.521	.02066	48.4121	.03812	26.2296	.05562	17.9802	49
12	.00349	286.478	.02095	47.7395	.03842	26.0307	.05591	17.8863	48
13	.00378	264.441	.02124	47.0853	.03871	25.8348	.05620	17.7934	47
14	.00407	245.552	.02153	46.4489	.03900	25.6418	.05649	17.7015	46
15	.00436	229.182	.02182	45.8294	.03929	25.4517	.05678	17.6106	45
16	.00465	214.858	.02211	45.2261	.03958	25.2644	.05708	17.5205	44
17	.00495	202.219	.02240	44.6386	.03987	25.0798	.05737	17.4314	43
18	.00524	190.984	.02269	44.0661	.04016	24.8978	.05766	17.3432	42
19	.00553	180.932	.02298	43.5081	.04046	24.7185	.05795	17.2558	41
20	.00582	171.885	.02328	42.9641	.04075	24.5418	.05824	17.1693	40
21	.00611	163.700	.02357	42.4335	.04104	24.3675	.05854	17.0837	39
22	.00640	156.259	.02386	41.9158	.04133	24.1957	.05883	16.9990	38
23	.00669	149.465	.02415	41.4106	.04162	24.0263	.05912	16.9150	37
24	.00698	143.237	.02444	40.9174	.04191	23.8593	.05941	16.8319	36
25	.00727	137.507	.02473	40.4358	.04220	23.6945	.05970	16.7496	35
26	.00756	132.219	.02502	39.9655	.04250	23.5321	.05999	16.6681	34
27	.00785	127.321	.02531	39.5059	.04279	23.3718	.06029	16.5874	33
28	.00815	122.774	.02560	39.0568	.04308	23.2137	.06058	16.5075	32
29	.00844	118.540	.02589	38.6177	.04337	23.0577	.06087	16.4283	31
30	.00873	114.589	.02619	38.1885	.04366	22.9038	.06116	16.3499	30
31	.00902	110.892	.02648	37.7686	.04395	22.7519	.06145	16.2722	29
32	.00931	107.426	.02677	37.3579	.04424	22.6020	.06175	16.1952	28
33	.00960	104.171	.02706	36.9560	.04454	22.4541	.06204	16.1190	27
34	.00989	101.107	.02735	36.5627	.04483	22.3081	.06233	16.0435	26
35	.01018	98.2179	.02764	36.1776	.04512	22.1640	.06262	15.9687	25
36	.01047	95.4895	.02793	35.8006	.04541	22.0217	.06291	15.8945	24
37	.01076	92.9085	.02822	35.4313	.04570	21.8813	.06321	15.8211	23
38	.01105	90.4633	.02851	35.0695	.04599	21.7426	.06350	15.7483	22
39	.01135	88.1436	.02881	34.7151	.04628	21.6056	.06379	15.6762	21
40	.01164	85.9398	.02910	34.3678	.04658	21.4704	.06408	15.6048	20
41	.01193	83.8435	.02939	34.0273	.04687	21.3369	.06437	15.5340	19
42	.01222	81.8470	.02968	33.6935	.04716	21.2049	.06467	15.4638	18
43	.01251	79.9434	.02997	33.3662	.04745	21.0747	.06496	15.3943	17
44	.01280	78.1263	.03026	33.0452	.04774	20.9460	.06525	15.3254	16
45	.01309	76.3900	.03055	32.7303	.04803	20.8188	.06554	15.2571	15
46	.01338	74.7292	.03084	32.4213	.04833	20.6932	.06584	15.1893	14
47	.01367	73.1390	.03114	32.1181	.04862	20.5691	.06613	15.1222	13
48	.01396	71.6151	.03143	31.8205	.04891	20.4465	.06642	15.0557	12
49	.01425	70.1533	.03172	31.5284	.04920	20.3253	.06671	14.9898	11
50	.01455	68.7501	.03201	31.2416	.04949	20.2056	.06700	14.9244	10
51	.01484	67.4019	.03230	30.9599	.04978	20.0872	.06730	14.8596	9
52	.01513	66.1055	.03259	30.6833	.05007	19.9702	.06759	14.7954	8
53	.01542	64.8580	.03288	30.4116	.05037	19.8546	.06788	14.7317	7
54	.01571	63.6567	.03317	30.1446	.05066	19.7403	.06817	14.6685	6
55	.01600	62.4992	.03346	29.8823	.05095	19.6273	.06847	14.6059	5
56	.01629	61.3829	.03376	29.6245	.05124	19.5156	.06876	14.5438	4
57	.01658	60.3058	.03405	29.3711	.05153	19.4051	.06905	14.4823	3
58	.01687	59.2659	.03434	29.1220	.05182	19.2959	.06934	14.4212	2
59	.01716	58.2612	.03463	28.8771	.05212	19.1879	.06963	14.3607	1
60	.01746	57.2900	.03492	28.6363	.05241	19.0811	.06993	14.3007	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	89°		88°		87°		86°		

′	4° Tang	4° Cotang	5° Tang	5° Cotang	6° Tang	6° Cotang	7° Tang	7° Cotang	′
0	.06993	14.3007	.08749	11.4301	.10510	9.51436	.12278	8.14435	60
1	.07022	14.2411	.08778	11.3919	.10540	9.48781	.12308	8.12481	59
2	.07051	14.1821	.08807	11.3540	.10569	9.46141	.12338	8.10536	58
3	.07080	14.1235	.08837	11.3163	.10599	9.43515	.12367	8.08600	57
4	.07110	14.0655	.08866	11.2789	.10628	9.40904	.12397	8.06674	56
5	.07139	14.0079	.08895	11.2417	.10657	9.38307	.12426	8.04756	55
6	.07168	13.9507	.08925	11.2048	.10687	9.35724	.12456	8.02848	54
7	.07197	13.8940	.08954	11.1681	.10716	9.33155	.12485	8.00948	53
8	.07227	13.8378	.08983	11.1316	.10746	9.30599	.12515	7.99058	52
9	.07256	13.7821	.09013	11.0954	.10775	9.28058	.12544	7.97176	51
10	.07285	13.7267	.09042	11.0594	.10805	9.25530	.12574	7.95302	50
11	.07314	13.6719	.09071	11.0237	.10834	9.23016	.12603	7.93438	49
12	.07344	13.6174	.09101	10.9882	.10863	9.20516	.12633	7.91582	48
13	.07373	13.5634	.09130	10.9529	.10893	9.18028	.12662	7.89734	47
14	.07402	13.5098	.09159	10.9178	.10922	9.15554	.12692	7.87895	46
15	.07431	13.4566	.09189	10.8829	.10952	9.13093	.12722	7.86064	45
16	.07461	13.4039	.09218	10.8483	.10981	9.10646	.12751	7.84242	44
17	.07490	13.3515	.09247	10.8139	.11011	9.08211	.12781	7.82428	43
18	.07519	13.2996	.09277	10.7797	.11040	9.05789	.12810	7.80622	42
19	.07548	13.2480	.09306	10.7457	.11070	9.03379	.12840	7.78825	41
20	.07578	13.1969	.09335	10.7119	.11099	9.00983	.12869	7.77035	40
21	.07607	13.1461	.09365	10.6783	.11128	8.98598	.12899	7.75254	39
22	.07636	13.0958	.09394	10.6450	.11158	8.96227	.12929	7.73480	38
23	.07665	13.0458	.09423	10.6118	.11187	8.93867	.12958	7.71715	37
24	.07695	12.9962	.09453	10.5789	.11217	8.91520	.12988	7.69957	36
25	.07724	12.9469	.09482	10.5462	.11246	8.89185	.13017	7.68208	35
26	.07753	12.8981	.09511	10.5136	.11276	8.86862	.13047	7.66466	34
27	.07782	12.8496	.09541	10.4813	.11305	8.84551	.13076	7.64732	33
28	.07812	12.8014	.09570	10.4491	.11335	8.82252	.13106	7.63005	32
29	.07841	12.7536	.09600	10.4172	.11364	8.79964	.13136	7.61287	31
30	.07870	12.7062	.09629	10.3854	.11394	8.77689	.13165	7.59575	30
31	.07899	12.6591	.09658	10.3538	.11423	8.75425	.13195	7.57872	29
32	.07929	12.6124	.09688	10.3224	.11452	8.73172	.13224	7.56176	28
33	.07958	12.5660	.09717	10.2913	.11482	8.70931	.13254	7.54487	27
34	.07987	12.5199	.09746	10.2602	.11511	8.68701	.13284	7.52806	26
35	.08017	12.4742	.09776	10.2294	.11541	8.66482	.13313	7.51132	25
36	.08046	12.4288	.09805	10.1988	.11570	8.64275	.13343	7.49465	24
37	.08075	12.3838	.09834	10.1683	.11600	8.62078	.13372	7.47806	23
38	.08104	12.3390	.09864	10.1381	.11629	8.59893	.13402	7.46154	22
39	.08134	12.2946	.09893	10.1080	.11659	8.57718	.13432	7.44509	21
40	.08163	12.2505	.09923	10.0780	.11688	8.55555	.13461	7.42871	20
41	.08192	12.2067	.09952	10.0483	.11718	8.53402	.13491	7.41240	19
42	.08221	12.1632	.09981	10.0187	.11747	8.51259	.13521	7.39616	18
43	.08251	12.1201	.10011	9.98931	.11777	8.49128	.13550	7.37999	17
44	.08280	12.0772	.10040	9.96007	.11806	8.47007	.13580	7.36389	16
45	.08309	12.0346	.10069	9.93101	.11836	8.44896	.13609	7.34786	15
46	.08339	11.9923	.10099	9.90211	.11865	8.42795	.13639	7.33190	14
47	.08368	11.9504	.10128	9.87338	.11895	8.40705	.13669	7.31600	13
48	.08397	11.9087	10158	9.84482	.11924	8.38625	.13698	7.30018	12
49	.08427	11.8673	.10187	9.81641	.11954	8.36555	.13728	7.28442	11
50	.08456	11.8262	.10216	9.78817	.11983	8.34496	.13758	7.26873	10
51	.08485	11.7853	.10246	9.76009	.12013	8.32446	.13787	7.25310	9
52	.08514	11.7448	.10275	9.73217	.12042	8.30406	.13817	7.23754	8
53	.08544	11.7045	.10305	9.70441	.12072	8.28376	.13846	7.22204	7
54	.08573	11.6645	.10334	9.67680	.12101	8.26355	.13876	7.20661	6
55	.08602	11.6248	.10363	9.64935	.12131	8.24345	.13906	7.19125	5
56	.08632	11.5853	.10393	9.62205	.12160	8.22344	.13935	7.17594	4
57	.08661	11.5461	.10422	9.59490	.12190	8.20352	.13965	7.16071	3
58	.08690	11.5072	.10452	9.56791	.12219	8.18370	.13995	7.14553	2
59	.08720	11.4685	.10481	9.54106	.12249	8.16398	.14024	7.13042	1
60	.08749	11.4301	.10510	9.51436	.12278	8.14435	.14054	7.11537	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	85°		84°		83°		82°		

′	8° Tang	8° Cotang	9° Tang	9° Cotang	10° Tang	10° Cotang	11° Tang	11° Cotang	′
0	.14054	7.11537	.15838	6.31375	.17633	5.67128	.19438	5.14455	60
1	.14084	7.10038	.15868	6.30189	.17663	5.66165	.19468	5.13658	59
2	.14113	7.08546	.15898	6.29007	.17693	5.65205	.19498	5.12862	58
3	.14143	7.07059	.15928	6.27829	.17723	5.64248	.19529	5.12069	57
4	.14173	7.05579	.15958	6.26655	.17753	5.63295	.19559	5.11279	56
5	.14202	7.04105	.15988	6.25486	.17783	5.62344	.19589	5.10490	55
6	.14232	7.02637	.16017	6.24321	.17813	5.61397	.19619	5.09704	54
7	.14262	7.01173	.16047	6.23160	.17843	5.60452	.19649	5.08921	53
8	.14291	6.99718	.16077	6.22003	.17873	5.59511	.19680	5.08139	52
9	.14321	6.98268	.16107	6.20851	.17903	5.58573	.19710	5.07360	51
10	.14351	6.96823	.16137	6.19703	.17933	5.57638	.19740	5.06584	50
11	.14381	6.95385	.16167	6.18559	.17963	5.56706	.19770	5.05809	49
12	.14410	6.93952	.16196	6.17419	.17993	5.55777	.19801	5.05037	48
13	.14440	6.92525	.16226	6.16283	.18023	5.54851	.19831	5.04267	47
14	.14470	6.91104	.16256	6.15151	.18053	5.53927	.19861	5.03499	46
15	.14499	6.89688	.16286	6.14023	.18083	5.53007	.19891	5.02734	45
16	.14529	6.88278	.16316	6.12899	.18113	5.52090	.19921	5.01971	44
17	.14559	6.86874	.16346	6.11779	.18143	5.51176	.19952	5.01210	43
18	.14588	6.85475	.16376	6.10664	.18173	5.50264	.19982	5.00451	42
19	.14618	6.84082	.16405	6.09552	.18203	5.49356	.20012	4.99695	41
20	.14648	6.82694	.16435	6.08444	.18233	5.48451	.20042	4.98940	40
21	.14678	6.81312	.16465	6.07340	.18262	5.47548	.20073	4.98188	39
22	.14707	6.79936	.16495	6.06240	.18293	5.46648	.20103	4.97438	38
23	.14737	6.78564	.16525	6.05143	.18323	5.45751	.20133	4.96690	37
24	.14767	6.77199	.16555	6.04051	.18353	5.44857	.20164	4.95945	36
25	.14796	6.75838	.16585	6.02962	.18384	5.43966	.20194	4.95201	35
26	.14826	6.74483	.16615	6.01878	.18414	5.43077	.20224	4.94460	34
27	.14856	6.73133	.16645	6.00797	.18444	5.42192	.20254	4.93721	33
28	.14886	6.71789	.16674	5.99720	.18474	5.41309	.20285	4.92984	32
29	.14915	6.70450	.16704	5.98646	.18504	5.40429	.20315	4.92249	31
30	.14945	6.69116	.16734	5.97576	.18534	5.39552	.20345	4.91516	30
31	.14975	6.67787	.16764	5.96510	.18564	5.38677	.20376	4.90785	29
32	.15005	6.66463	.16794	5.95448	.18594	5.37805	.20406	4.90056	28
33	.15034	6.65144	.16824	5.94390	.18624	5.36936	.20436	4.89330	27
34	.15064	6.63831	.16854	5.93335	.18654	5.36070	.20466	4.88605	26
35	.15094	6.62523	.16884	5.92283	.18684	5.35206	.20497	4.87882	25
36	.15124	6.61219	.16914	5.91236	.18714	5.34345	.20527	4.87162	24
37	.15153	6.59921	.16944	5.90191	.18745	5.33487	.20557	4.86444	23
38	.15183	6.58627	.16974	5.89151	.18775	5.32631	.20588	4.85727	22
39	.15213	6.57339	.17004	5.88114	.18805	5.31778	.20618	4.85013	21
40	.15243	6.56055	.17033	5.87080	.18835	5.30928	.20648	4.84300	20
41	.15272	6.54777	.17063	5.86051	.18865	5.30080	.20679	4.83590	19
42	.15302	6.53503	.17093	5.85024	.18895	5.29235	.20709	4.82882	18
43	.15332	6.52234	.17123	5.84001	.18925	5.28393	.20739	4.82175	17
44	.15362	6.50970	.17153	5.82982	.18955	5.27553	.20770	4.81471	16
45	.15391	6.49710	.17183	5.81966	.18986	5.26715	.20800	4.80769	15
46	.15421	6.48456	.17213	5.80953	.19016	5.25880	.20830	4.80068	14
47	.15451	6.47206	.17243	5.79944	.19046	5.25048	.20861	4.79370	13
48	.15481	6.45961	.17273	5.78938	.19076	5.24218	.20891	4.78673	12
49	.15511	6.44720	.17303	5.77936	.19106	5.23391	.20921	4.77978	11
50	.15540	6.43484	.17333	5.76937	.19136	5.22566	.20952	4.77286	10
51	.15570	6.42253	.17363	5.75941	.19166	5.21744	.20982	4.76595	9
52	.15600	6.41026	.17393	5.74949	.19197	5.20925	.21013	4.75906	8
53	.15630	6.39804	.17423	5.73960	.19227	5.20107	.21043	4.75219	7
54	.15660	6.38587	.17453	5.72974	.19257	5.19293	.21073	4.74534	6
55	.15689	6.37374	.17483	5.71992	.19287	5.18480	.21104	4.73851	5
56	.15719	6.36165	.17513	5.71013	.19317	5.17671	.21134	4.73170	4
57	.15749	6.34961	.17543	5.70037	.19347	5.16863	.21164	4.72490	3
58	.15779	6.33761	.17573	5.69064	.19378	5.16058	.21195	4.71813	2
59	.15809	6.32566	.17603	5.68094	.19408	5.15256	.21225	4.71137	1
60	.15838	6.31375	.17633	5.67128	.19438	5.14455	.21256	4.70463	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	81°		80°		79°		78°		

′	12°		13°		14°		15°		′
	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	
0	.21256	4.70463	.23087	4.33148	.24933	4.01078	.26795	3.73205	60
1	.21286	4.69791	.23117	4.32573	.24964	4.00582	.26826	3.72771	59
2	.21316	4.69121	.23148	4.32001	.24995	4.00086	.26857	3.72338	58
3	.21347	4.68452	.23179	4.31430	.25026	3.99592	.26888	3.71907	57
4	.21377	4.67786	.23209	4.30860	.25056	3.99099	.26920	3.71476	56
5	.21408	4.67121	.23240	4.30291	.25087	3.98607	.26951	3.71046	55
6	.21438	4.66458	.23271	4.29724	.25118	3.98117	.26982	3.70616	54
7	.21469	4.65797	.23301	4.20159	.25149	3.97627	.27013	3.70188	53
8	.21499	4.65138	.23332	4.28595	.25180	3.97139	.27044	3.69761	52
9	.21529	4.64480	.23363	4.28032	.25211	3.96651	.27076	3.69335	51
10	.21560	4.63825	.23393	4.27471	.25242	3.96165	.27107	3.68909	50
11	.21590	4.63171	.23424	4.26911	.25273	3.95680	.27138	3.68485	49
12	.21621	4.62518	.23455	4.26352	.25304	3.95196	.27169	3.68061	48
13	.21651	4.61868	.23485	4.25795	.25335	3.94713	.27201	3.67638	47
14	.21682	4.61219	.23516	4.25239	.25366	3.94232	.27232	3.67217	46
15	.21712	4.60572	.23547	4.24685	.25397	3.93751	.27263	3.66796	45
16	.21743	4.59927	.23578	4.24132	.25428	3.93271	.27294	3.66376	44
17	.21773	4.59283	.23608	4.23580	.25459	3.92793	.27326	3.65957	43
18	.21804	4.58641	.23639	4.23030	.25490	3.92316	.27357	3.65538	42
19	.21834	4.58001	.23670	4.22481	.25521	3.91839	.27388	3.65121	41
20	.21864	4.57363	.23700	4.21933	.25552	3.91364	.27419	3.64705	40
21	.21895	4.56726	.23731	4.21387	.25583	3.90890	.27451	3.64289	39
22	.21925	4.56091	.23762	4.20842	.25614	3.90417	.27482	3.63874	38
23	.21956	4.55458	.23793	4.20298	.25645	3.89945	.27513	3.63461	37
24	.21986	4.54826	.23823	4.19756	.25676	3.89474	.27545	3.63048	36
25	.22017	4.54196	.23854	4.19215	.25707	3.89004	.27576	3.62636	35
26	.22047	4.53568	.23885	4.18675	.25738	3.88536	.27607	3.62224	34
27	.22078	4.52941	.23916	4.18137	.25769	3.88068	.27638	3.61814	33
28	.22108	4.52316	.23946	4.17600	.25800	3.87601	.27670	3.61405	32
29	.22139	4.51693	.23977	4.17064	.25831	3.87136	.27701	3.60996	31
30	.22169	4.51071	.24008	4.16530	.25862	3.86671	.27732	3.60588	30
31	.22200	4.50451	.24039	4.15997	.25893	3.86208	.27764	3.60181	29
32	.22231	4.49832	.24069	4.15465	.25924	3.85745	.27795	3.59775	28
33	.22261	4.49215	.24100	4.14934	.25955	3.85284	.27826	3.59370	27
34	.22292	4.48600	.24131	4.14405	.25986	3.84824	.27858	3.58966	26
35	.22322	4.47986	.24162	4.13877	.26017	3.84364	.27889	3.58562	25
36	.22353	4.47374	.24193	4.13350	.26048	3.83906	.27921	3.58160	24
37	.22383	4.46764	.24223	4.12825	.26079	3.83449	.27952	3.57758	23
38	.22414	4.46155	.24254	4.12301	.26110	3.82992	.27983	3.57357	22
39	.22444	4.45548	.24285	4.11778	.26141	3.82537	.28015	3.56957	21
40	.22475	4.44942	.24316	4.11256	.26172	3.82083	.28046	3.56557	20
41	.22505	4.44338	.24347	4.10736	.26203	3.81630	.28077	3.56159	19
42	.22536	4.43735	.24377	4.10216	.26235	3.81177	.28109	3.55761	18
43	.22567	4.43134	.24408	4.09699	.26266	3.80726	.28140	3.55364	17
44	.22597	4.42534	.24439	4.09182	.26297	3.80276	.28172	3.54968	16
45	.22628	4.41936	.24470	4.08666	.26328	3.79827	.28203	3.54573	15
46	.22658	4.41340	.24501	4.08152	.26359	3.79378	.28234	3.54179	14
47	.22689	4.40745	.24532	4.07639	.26390	3.78931	.28266	3.53785	13
48	.22719	4.40152	.24562	4.07127	.26421	3.78485	.28297	3.53393	12
49	.22750	4.39560	.24593	4.06616	.26452	3.78040	.28329	3.53001	11
50	.22781	4.38969	.24624	4.06107	.26483	3.77595	.28360	3.52609	10
51	.22811	4.38381	.24655	4.05599	.26515	3.77152	.28391	3.52219	9
52	.22842	4.37793	.24686	4.05092	.26546	3.76709	.28423	3.51829	8
53	.22872	4.37207	.24717	4.04586	.26577	3.76268	.28454	3.51441	7
54	.22903	4.36623	.24747	4.04081	.26608	3.75828	.28486	3.51053	6
55	.22934	4.36040	.24778	4.03578	.26639	3.75388	.28517	3.50666	5
56	.22964	4.35459	.24809	4.03076	.26670	3.74950	.28549	3.50279	4
57	.22995	4.34879	.24840	4.02574	.26701	3.74512	.28580	3.49894	3
58	.23026	4.34300	.24871	4.02074	.26733	3.74075	.28612	3.49509	2
59	.23056	4.33723	.24902	4.01576	.26764	3.73640	.28643	3.49125	1
60	.23087	4.33148	.24933	4.01078	.26795	3.73205	.28675	3.48741	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	77°		76°		75°		74°		

	16°		17°		18°		19°		
′	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	′
0	.28675	3.48741	.30573	3.27085	.32492	3.07768	.34433	2.90421	60
1	.28706	3.48359	.30605	3.26745	.32524	3.07464	.34465	2.90147	59
2	.28738	3.47977	.30637	3.26406	.32556	3.07160	.34498	2.89873	58
3	.28769	3.47596	.30669	3.26067	.32588	3.06857	.34530	2.89600	57
4	.28800	3.47216	.30700	3.25729	.32621	3.06554	.34563	2.89327	56
5	.28832	3.46837	.30732	3.25392	.32653	3.06252	.34596	2.89055	55
6	.28864	3.46458	.30764	3.25055	.32685	3.05950	.34628	2.88783	54
7	.28895	3.46080	.30796	3.24719	.32717	3.05649	.34661	2.88511	53
8	.28927	3.45703	.30828	3.24383	.32749	3.05349	.34693	2.88240	52
9	.28958	3.45327	.30860	3.24049	.32782	3.05049	.34726	2.87970	51
10	.28990	3.44951	.30891	3.23714	.32814	3.04749	.34758	2.87700	50
11	.29021	3.44576	.30923	3.23381	.32846	3.04450	.34791	2.87430	49
12	.29053	3.44202	.30955	3.23048	.32878	3.04152	.34824	2.87161	48
13	.29084	3.43829	.30987	3.22715	.32911	3.03854	.34856	2.86892	47
14	.29116	3.43456	.31019	3.22384	.32943	3.03556	.34889	2.86624	46
15	.29147	3.43084	.31051	3.22053	.32975	3.03260	.34922	2.86356	45
16	.29179	3.42713	.31083	3.21722	.33007	3.02963	.34954	2.86089	44
17	.29210	3.42343	.31115	3.21392	.33040	3.02667	.34987	2.85822	43
18	.29242	3.41973	.31147	3.21063	.33072	3.02372	.35020	2.85555	42
19	.29274	3.41604	.31178	3.20734	.33104	3.02077	.35052	2.85289	41
20	.29305	3.41236	.31210	3.20406	.33136	3.01783	.35085	2.85023	40
21	.29337	3.40869	.31242	3.20079	.33169	3.01489	.35118	2.84758	39
22	.29368	3.40502	.31274	3.19752	.33201	3.01196	.35150	2.84494	38
23	.29400	3.40136	.31306	3.19426	.33233	3.00903	.35183	2.84229	37
24	.29432	3.39771	.31338	3.19100	.33266	3.00611	.35216	2.83965	36
25	.29463	3.39406	.31370	3.18775	.33298	3.00319	.35248	2.83702	35
26	.29495	3.39042	.31402	3.18451	.33330	3.00028	.35281	2.83439	34
27	.29526	3.38679	.31434	3.18127	.33363	2.99738	.35314	2.83176	33
28	.29558	3.38317	.31466	3.17804	.33395	2.99447	.35346	2.82914	32
29	.29590	3.37955	.31498	3.17481	.33427	2.99158	.35379	2.82653	31
30	.29621	3.37594	.31530	3.17159	.33460	2.98868	.35412	2.82391	30
31	.29653	3.37234	.31562	3.16838	.33492	2.98580	.35445	2.82130	29
32	.29685	3.36875	.31594	3.16517	.33524	2.98292	.35477	2.81870	28
33	.29716	3.36516	.31626	3.16197	.33557	2.98004	.35510	2.81610	27
34	.29748	3.36158	.31658	3.15877	.33589	2.97717	.35543	2.81350	26
35	.29780	3.35800	.31690	3.15558	.33621	2.97430	.35576	2.81091	25
36	.29811	3.35443	.31722	3.15240	.33654	2.97144	.35608	2.80833	24
37	.29843	3.35087	.31754	3.14922	.33686	2.96858	.35641	2.80574	23
38	.29875	3.34732	.31786	3.14605	.33718	2.96573	.35674	2.80316	22
39	.29906	3.34377	.31818	3.14288	.33751	2.96288	.35707	2.80059	21
40	.29938	3.34023	.31850	3.13972	.33783	2.96004	.35740	2.79802	20
41	.29970	3.33670	.31882	3.13656	.33816	2.95721	.35772	2.79545	19
42	.30001	3.33317	.31914	3.13341	.33848	2.95437	.35805	2.79289	18
43	.30033	3.32965	.31946	3.13027	.33881	2.95155	.35838	2.79033	17
44	.30065	3.32614	.31978	3.12713	.33913	2.94872	.35871	2.78778	16
45	.30097	3.32264	.32010	3.12400	.33945	2.94591	.35904	2.78523	15
46	.30128	3.31914	.32042	3.12087	.33978	2.94309	.35937	2.78269	14
47	.30160	3.31565	.32074	3.11775	.34010	2.94028	.35969	2.78014	13
48	.30192	3.31216	.32106	3.11464	.34043	2.93748	.36002	2.77761	12
49	.30224	3.30868	.32139	3.11153	.34075	2.93468	.36035	2.77507	11
50	.30255	3.30521	.32171	3.10842	.34108	2.93189	.36068	2.77254	10
51	.30287	3.30174	.32203	3.10532	.34140	2.92910	.36101	2.77002	9
52	.30319	3.29829	.32235	3.10223	.34173	2.92632	.36134	2.76750	8
53	.30351	3.29483	.32267	3.09914	.34205	2.92354	.36167	2.76498	7
54	.30382	3.29139	.32299	3.09606	.34238	2.92076	.36199	2.76247	6
55	.30414	3.28795	.32331	3.09298	.34270	2.91799	.36232	2.75996	5
56	.30446	3.28452	.32363	3.08991	.34303	2.91523	.36265	2.75746	4
57	.30478	3.28109	.32396	3.08685	.34335	2.91246	.36298	2.75496	3
58	.30509	3.27767	.32428	3.08379	.34368	2.90971	.36331	2.75246	1
59	.30541	3.27426	.32460	3.08073	.34400	2.90696	.36364	2.74997	0
60	.30573	3.27085	.32492	3.07768	.34433	2.90421	.36397	2.74748	2
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	73°		72°		71°		70°		

′	20°		21°		22°		23°		′
	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	
0	.36397	2.74748	.38386	2.60509	.40403	2.47509	.42447	2.35585	60
1	.36430	2.74499	.38420	2.60283	.40436	2.47302	.42482	2.35395	59
2	.36463	2.74251	.38453	2.60057	.40470	2.47095	.42516	2.35205	58
3	.36496	2.74004	.38487	2.59831	.40504	2.46888	.42551	2.35015	57
4	.36529	2.73756	.38520	2.59606	.40538	2.46682	.42585	2.34825	56
5	.36562	2.73509	.38553	2.59381	.40572	2.46476	.42619	2.34636	55
6	.36595	2.73263	.38587	2.59156	.40606	2.46270	.42654	2.34447	54
7	.36628	2.73017	.38620	2.58932	.40640	2.46065	.42688	2.34258	53
8	.36661	2.72771	.38654	2.58708	.40674	2.45860	.42722	2.34069	52
9	.36694	2.72526	.38687	2.58484	.40707	2.45655	.42757	2.33881	51
10	.36727	2.72281	.38721	2.58261	.40741	2.45451	.42791	2.33693	50
11	.36760	2.72036	.38754	2.58038	.40775	2.45246	.42826	2.33505	49
12	.36793	2.71792	.38787	2.57815	.40809	2.45043	.42860	2.33317	48
13	.36826	2.71548	.38821	2.57593	.40843	2.44839	.42894	2.33130	47
14	.36859	2.71305	.38854	2.57371	.40877	2.44636	.42929	2.32943	46
15	.36892	2.71062	.38888	2.57150	.40911	2.44433	.42963	2.32756	45
16	.36925	2.70819	.38921	2.56928	.40945	2.44230	.42998	2.32570	44
17	.36958	2.70577	.38955	2.56707	.40979	2.44027	.43032	2.32383	43
18	.36991	2.70335	.38988	2.56487	.41013	2.43825	.43067	2.32197	42
19	.37024	2.70094	.39022	2.56266	.41047	2.43623	.43101	2.32012	41
20	.37057	2.69853	.39055	2.56046	.41081	2.43422	.43136	2.31826	40
21	.37090	2.69612	.39089	2.55827	.41115	2.43220	.43170	2.31641	39
22	.37123	2.69371	.39122	2.55608	.41149	2.43019	.43205	2.31456	38
23	.37157	2.69131	.39156	2.55389	.41183	2.42819	.43239	2.31271	37
24	.37190	2.68892	.39190	2.55170	.41217	2.42618	.43274	2.31086	36
25	.37223	2.68653	.39223	2.54952	.41251	2.42418	.43308	2.30902	35
26	.37256	2.68414	.39257	2.54734	.41285	2.42218	.43343	2.30718	34
27	.37289	2.68175	.39290	2.54516	.41319	2.42019	.43378	2.30534	33
28	.37322	2.67937	.39324	2.54299	.41353	2.41819	.43412	2.30351	32
29	.37355	2.67700	.39357	2.54082	.41387	2.41620	.43447	2.30167	31
30	.37388	2.67462	.39391	2.53865	.41421	2.41421	.43481	2.29984	30
31	.37422	2.67225	.39425	2.53648	.41455	2.41223	.43516	2.29801	29
32	.37455	2.66989	.39458	2.53432	.41490	2.41025	.43550	2.29619	28
33	.37488	2.66752	.39492	2.53217	.41524	2.40827	.43585	2.29437	27
34	.37521	2.66516	.39526	2.53001	.41558	2.40629	.43620	2.29254	26
35	.37554	2.66281	.39559	2.52786	.41592	2.40432	.43654	2.29073	25
36	.37588	2.66046	.39593	2.52571	.41626	2.40235	.43689	2.28891	24
37	.37621	2.65811	.39626	2.52357	.41660	2.40038	.43724	2.28710	23
38	.37654	2.65576	.39660	2.52142	.41694	2.39841	.43758	2.28528	22
39	.37687	2.65342	.39694	2.51929	.41728	2.39645	.43793	2.28348	21
40	.37720	2.65109	.39727	2.51715	.41763	2.39449	.43828	2.28167	20
41	.37754	2.64875	.39761	2.51502	.41797	2.39253	.43862	2.27987	19
42	.37787	2.64642	.39795	2.51289	.41831	2.39058	.43897	2.27806	18
43	.37820	2.64410	.39829	2.51076	.41865	2.38863	.43932	2.27626	17
44	.37853	2.64177	.39862	2.50864	.41899	2.38668	.43966	2.27447	16
45	.37887	2.63945	.39896	2.50652	.41933	2.38473	.44001	2.27267	15
46	.37920	2.63714	.39930	2.50440	.41968	2.38279	.44036	2.27088	14
47	.37953	2.63483	.39963	2.50229	.42002	2.38084	.44071	2.26909	13
48	.37986	2.63252	.39997	2.50018	.42036	2.37891	.44105	2.26730	12
49	.38020	2.63021	.40031	2.49807	.42070	2.37697	.44140	2.26552	11
50	.38053	2.62791	.40065	2.49597	.42105	2.37504	.44175	2.26374	10
51	.38086	2.62561	.40098	2.49386	.42139	2.37311	.44210	2.26196	9
52	.38120	2.62332	.40132	2.49177	.42173	2.37118	.44244	2.26018	8
53	.38153	2.62103	.40166	2.48967	.42207	2.36925	.44279	2.25840	7
54	.38186	2.61874	.40200	2.48758	.42242	2.36733	.44314	2.25663	6
55	.38220	2.61646	.40234	2.48549	.42276	2.36541	.44349	2.25486	5
56	.38253	2.61418	.40267	2.48340	.42310	2.36349	.44384	2.25309	4
57	.38286	2.61190	.40301	2.48132	.42345	2.36158	.44418	2.25132	3
58	.38320	2.60963	.40335	2.47924	.42379	2.35967	.44453	2.24956	2
59	.38353	2.60736	.40369	2.47716	.42413	2.35776	.44488	2.24780	1
60	.38386	2.60509	.40403	2.47509	.42447	2.35585	.44523	2.24604	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	69°		68°		67°		66°		

′	24°		25°		26°		27°		′
	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	
0	.44523	2.24604	.46631	2.14451	.48773	2.05030	.50953	1.96261	60
1	.44558	2.24428	.46666	2.14288	.48809	2.04879	.50989	1.96120	59
2	.44593	2.24252	.46702	2.14125	.48845	2.04728	.51026	1.95979	58
3	.44627	2.24077	.46737	2.13963	.48881	2.04577	.51063	1.95838	57
4	.44662	2.23902	.46772	2.13801	.48917	2.04426	.51099	1.95698	56
5	.44697	2.23727	.46808	2.13639	.48953	2.04276	.51136	1.95557	55
6	.44732	2.23553	.46843	2.13477	.48989	2.04125	.51173	1.95417	54
7	.44767	2.23378	.46879	2.13316	.49026	2.03975	.51209	1.95277	53
8	.44802	2.23204	.46914	2.13154	.49062	2.03825	.51246	1.95137	52
9	.44837	2.23030	.46950	2.12993	.49098	2.03675	.51283	1.94997	51
10	.44872	2.22857	.46985	2.12832	.49134	2.03526	.51319	1.94858	50
11	.44907	2.22683	.47021	2.12671	.49170	2.03376	.51356	1.94718	49
12	.44942	2.22510	.47056	2.12511	.49206	2.03227	.51393	1.94579	48
13	.44977	2.22337	.47092	2.12350	.49242	2.03078	.51430	1.94440	47
14	.45012	2.22164	.47128	2.12190	.49278	2.02929	.51467	1.94301	46
15	.45047	2.21992	.47163	2.12030	.49315	2.02780	.51503	1.94162	45
16	.45082	2.21819	.47199	2.11871	.49351	2.02631	.51540	1.94023	44
17	.45117	2.21647	.47234	2.11711	.49387	2.02483	.51577	1.93885	43
18	.45152	2.21475	.47270	2.11552	.49423	2.02335	.51614	1.93746	42
19	.45187	2.21304	.47305	2.11392	.48459	2.02187	.51651	1.93608	41
20	.45222	2.21132	.47341	2.11233	.49495	2.02039	51688	1.93470	40
21	.45257	2.20961	.47377	2.11075	.49532	2.01891	.51724	1.93332	39
22	.45292	2.20790	.47412	2.10916	.49568	2.01743	.51761	1.93195	38
23	.45327	2.20619	.47448	2.10758	.49604	2.01596	.51798	1.93057	37
24	.45362	2.20449	.47483	2.10600	.49640	2.01449	.51835	1.92920	36
25	.45397	2.20278	.47519	2.10442	.49677	2.01302	.51872	1.92782	35
26	.45432	2.20108	.47555	2.10284	.49713	2.01155	.51909	1.92645	34
27	.45467	2.19938	.47590	2.10126	.49749	2.01008	.51946	1.92508	33
28	.45502	2.19769	.47626	2.09969	.49786	2.00862	.51983	1.92371	32
29	.45538	2.19599	.47662	2.09811	.49822	2.00715	.52020	1.92235	31
30	.45573	2.19430	.47698	2.09654	.49858	2.00569	.52057	1.92098	30
31	.45608	2.19261	.47733	2.09498	.49894	2.00423	.52094	1.91962	29
32	.45643	2.19092	.47769	2.09341	.49931	2.00277	.52131	1.91826	28
33	.45678	2.18923	.47805	2.09184	.49967	2.00131	.52168	1.91690	27
34	.45713	2.18755	.47840	2.09028	.50004	1.99986	.52205	1.91552	26
35	.45748	2.18587	.47876	2.08872	.50040	1.99841	.52242	1.91414	25
36	.45784	2.18419	.47912	2.08716	.50076	1.99695	.52279	1.91288	24
37	.45819	2.18251	.47948	2.08560	.50113	1.99550	.52316	1.91142	23
38	.45854	2.18084	.47984	2.08405	.50149	1.99406	.52353	1.91017	22
39	.45889	2.17916	.48019	2.08250	.50185	1.99261	.52390	1.90876	21
40	.45924	2.17749	.48055	2.08094	.50222	1.99116	.52427	1.90741	20
41	.45960	2.17582	.48091	2.07939	.50258	1.98972	.52464	1.90607	19
42	.45995	2.17416	.48127	2.07785	.50295	1.98828	.52501	1.90472	18
43	.46030	2.17249	.48163	2.07630	.50331	1.98684	.52538	1.90337	17
44	.46065	2.17083	.48198	2.07476	.50368	1.98540	.52575	1.90203	16
45	.46101	2.16917	.48234	2.07321	.50404	1.98396	.52613	1.90069	15
46	.46136	2.16751	.48270	2.07167	.50441	1.98253	.52650	1.89935	14
47	.46171	2.16585	.48306	2.07014	.50477	1.98110	.52687	1.89801	13
48	.46206	2.16420	.48342	2.06860	.50514	1.97966	.52724	1.89667	12
49	.46242	2.16255	.48378	2.06706	.50550	1.97823	.52761	1.89533	11
50	.46277	2.16090	.48414	2.06553	.50587	1.97681	.52798	1.89400	10
51	.46312	2.15925	.48450	2.06400	.50623	1.97538	.52836	1.89266	9
52	.46348	2.15760	.48486	2.06247	.50660	1.97395	.52873	1.89133	8
53	.46383	2.15596	.48521	2.06094	.50696	1.97253	.52910	1.89000	7
54	.46418	2.15432	.48557	2.05942	.50733	1.97111	.52947	1.88867	6
55	.46454	2.15268	.48593	2.05790	.50769	1.96969	.52985	1.88734	5
56	.46489	2.15104	.48629	2.05637	.50806	1.96827	.53022	1.88602	4
57	.46525	2.14940	.48665	2.05485	.50843	1.96685	.53059	1.88469	3
58	.46560	2.14777	.48701	2.05333	.50879	1.96544	.53096	1.88337	2
59	.46595	2.14614	.48737	2.05182	.50916	1.96402	.53134	1.88205	1
60	.46631	2.14451	.48773	2.05030	.50953	1.96261	.53171	1.88073	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	65°		64°		63°		62°		

′	28° Tang	28° Cotang	29° Tang	29° Cotang	30° Tang	30° Cotang	31° Tang	31° Cotang	′
0	.53171	1.88073	.55431	1.80405	.57735	1.73205	.60086	1.66428	60
1	.53208	1.87941	.55469	1.80281	.57774	1.73089	.60126	1.66318	59
2	.53246	1.87809	.55507	1.80158	.57813	1.72973	.60165	1.66209	58
3	.53283	1.87677	.55545	1.80034	.57851	1.72857	.60205	1.66099	57
4	.53320	1.87546	.55583	1.79911	.57890	1.72741	.60245	1.65990	56
5	.53358	1.87415	.55621	1.79788	.57929	1.72625	.60284	1.65881	55
6	.53395	1.87283	.55659	1.79665	.57968	1.72509	.60324	1.65772	54
7	.53432	1.87152	.55697	1.79542	.58007	1.72393	.60364	1.65663	53
8	.53470	1.87021	.55736	1.79419	.58046	1.72278	.60403	1.65554	52
9	.53507	1.86891	.55774	1.79296	.58085	1.72163	.60443	1.65445	51
10	.53545	1.86760	.55812	1.79174	.58124	1.72047	.60483	1.65337	50
11	.53582	1.86630	.55850	1.79051	.58162	1.71932	.60522	1.65228	49
12	.53620	1.86499	.55888	1.78929	.58201	1.71817	.60562	1.65120	48
13	.53657	1.86369	.55926	1.78807	.58240	1.71702	.60602	1.65011	47
14	.53694	1.86239	.55964	1.78685	.58279	1.71588	.60642	1.64903	46
15	.53732	1.86109	.56003	1.78563	.58318	1.71473	.60681	1.64795	45
16	.53769	1.85979	.56041	1.78441	.58357	1.71358	.60721	1.64687	44
17	.53807	1.85850	.56079	1.78319	.58396	1.71244	.60761	1.64579	43
18	.53844	1.85720	.56117	1.78198	.58435	1.71129	.60801	1.64471	42
19	.53882	1.85591	.56156	1.78077	.58474	1.71015	.60841	1.64363	41
20	.53920	1.85462	.56194	1.77955	.58513	1.70901	.60881	1.64256	40
21	.53957	1.85333	.56232	1.77834	.58552	1.70787	.60921	1.64148	39
22	.53995	1.85204	.56270	1.77713	.58591	1.70673	.60960	1.64041	38
23	.54032	1.85075	.56309	1.77592	.58631	1.70560	.61000	1.63934	37
24	.54070	1.84946	.56347	1.77471	.58670	1.70446	.61040	1.63826	36
25	.54107	1.84818	.56385	1.77351	.58709	1.70332	.61080	1.63719	35
26	.54145	1.84689	.56424	1.77230	.58748	1.70219	.61120	1.63612	34
27	.54183	1.84561	.56462	1.77110	.58787	1.70106	.61160	1.63505	33
28	.54220	1.84433	.56501	1.76990	.58826	1.69992	.61200	1.63398	32
29	.54258	1.84305	.56539	1.76869	.58865	1.69879	.61240	1.63292	31
30	.54296	1.84177	.56577	1.76749	.58905	1.69766	.61280	1.63185	30
31	.54333	1.84049	.56616	1.76629	.58944	1.69653	.61320	1.63079	29
32	.54371	1.83922	.56654	1.76510	.58983	1.69541	.61360	1.62972	28
33	.54409	1.83794	.56693	1.76390	.59022	1.69428	.61400	1.62866	27
34	.54446	1.83667	.56731	1.76271	.59061	1.69316	.61440	1.62760	26
35	.54484	1.83540	.56769	1.76151	.59101	1.69203	.61480	1.62654	25
36	.54522	1.83413	.56808	1.76032	.59140	1.69091	.61520	1.62548	24
37	.54560	1.83286	.56846	1.75913	.59179	1.68979	.61561	1.62442	23
38	.54597	1.83159	.56885	1.75794	.59218	1.68866	.61601	1.62336	22
39	.54635	1.83033	.56923	1.75675	.59258	1.68754	.61641	1.62230	21
40	.54673	1.82906	.56962	1.75556	.59297	1.68643	.61681	1.62125	20
41	.54711	1.82780	.57000	1.75437	.59336	1.68531	.61721	1.62019	19
42	.54748	1.82654	.57039	1.75319	.59376	1.68419	.61761	1.61914	18
43	.54786	1.82528	.57078	1.75200	.59415	1.68308	.61801	1.61808	17
44	.54824	1.82402	.57116	1.75082	.59454	1.68196	.61842	1.61703	16
45	.54862	1.82276	.57155	1.74964	.59494	1.68085	.61882	1.61598	15
46	.54900	1.82150	.57193	1.74846	.59533	1.67974	.61922	1.61493	14
47	.54938	1.82025	.57232	1.74728	.59573	1.67863	.61962	1.61388	13
48	.54975	1.81899	.57271	1.74610	.59612	1.67752	.62003	1.61283	12
49	.55013	1.81774	.57309	1.74492	.59651	1.67641	.62043	1.61179	11
50	.55051	1.81649	.57348	1.74375	.59691	1.67530	.62083	1.61074	10
51	.55089	1.81524	.57386	1.74257	.59730	1.67419	.62124	1.60970	9
52	.55127	1.81399	.57425	1.74140	.59770	1.67309	.62164	1.60865	8
53	.55165	1.81274	.57464	1.74022	.59809	1.67198	.62204	1.60761	7
54	.55203	1.81150	.57503	1.73905	.59849	1.67088	.62245	1.60657	6
55	.55241	1.81025	.57541	1.73788	.59888	1.66978	.62285	1.60553	5
56	.55279	1.80901	.57580	1.73671	.59928	1.66867	.62325	1.60449	4
57	.55317	1.80777	.57619	1.73555	.59967	1.66757	.62366	1.60345	3
58	.55355	1.80653	.57657	1.73438	.60007	1.66647	.62406	1.60241	2
59	.55393	1.80529	.57696	1.73321	.60046	1.66538	.62446	1.60137	1
60	.55431	1.80405	.57735	1.73205	.60086	1.66428	.62487	1.60033	0
′	61° Cotang	61° Tang	60° Cotang	60° Tang	59° Cotang	59° Tang	58° Cotang	58° Tang	′

′	32° Tang	32° Cotang	33° Tang	33° Cotang	34° Tang	34° Cotang	35° Tang	35° Cotang	′
0	.62487	1.60033	.64941	1.53986	.67451	1.48256	.70021	1.42815	60
1	.62527	1.59930	.64982	1.53888	.67493	1.48163	.70064	1.42726	59
2	.62568	1.59826	.65024	1.53791	.67536	1.48070	.70170	1.42638	58
3	.62608	1.59723	.65065	1.53693	.67578	1.47977	.70151	1.42550	57
4	.62649	1.59620	.65106	1.53595	.67620	1.47885	.70194	1.42462	56
5	.62689	1.59517	.65148	1.53497	.67663	1.47792	.70238	1.42374	55
6	.62730	1.59414	.65189	1.53400	.67705	1.47699	.70281	1.42286	54
7	.62770	1.59311	.65231	1.53302	.67748	1.47607	.70325	1.42198	53
8	.62811	1.59208	.65272	1.53205	.67790	1.47514	.70368	1.42110	52
9	.62852	1.59105	.65314	1.53107	.67832	1.47422	.70412	1.42022	51
10	.62892	1.59002	.65355	1.53010	.67875	1.47330	.70455	1.41934	50
11	.62933	1.58900	.65397	1.52913	.67917	1.47238	.70499	1.41847	49
12	.62973	1.58797	.65438	1.52816	.67960	1.47146	.70542	1.41759	48
13	.63014	1.58695	.65480	1.52719	.68002	1.47053	.70586	1.41672	47
14	.63055	1.58593	.65521	1.52622	.68045	1.46962	.70629	1.41584	46
15	.63095	1.58490	.65563	1.52525	.68088	1.46870	.70673	1.41497	45
16	.63136	1.58388	.65604	1.52429	.68130	1.46778	.70717	1.41409	44
17	.63177	1.58286	.65646	1.52332	.68173	1.46686	.70760	1.41322	43
18	.63217	1.58184	.65688	1.52235	.68215	1.46595	.70804	1.41235	42
19	.63258	1.58083	.65729	1.52139	.68258	1.46503	.70848	1.41148	41
20	.63299	1.57981	.65771	1.52043	.68301	1.46411	.70891	1.41061	40
21	.63340	1.57879	.65813	1.51946	.68343	1.46320	.70935	1.40974	39
22	.63380	1.57778	.65854	1.51850	.68386	1.46229	.70979	1.40887	38
23	.63421	1.57676	.65896	1.51754	.68429	1.46137	.71023	1.40800	37
24	.63462	1.57575	.65938	1.51658	.68471	1.46046	.71066	1.40714	36
25	.63503	1.57474	.65980	1.51562	.68514	1.45955	.71110	1.40627	35
26	.63544	1.57372	.66021	1.51466	.68557	1.45864	.71154	1.40540	34
27	.63584	1.57271	.66063	1.51370	.68600	1.45773	.71198	1.40454	33
28	.63625	1.57170	.66105	1.51275	.68642	1.45682	.71242	1.40367	32
29	.63666	1.57069	.66147	1.51179	.68685	1.45592	.71285	1.40281	31
30	.63707	1.56969	.66189	1.51084	.68728	1.45501	.71329	1.40195	30
31	.63748	1.56868	.66230	1.50988	.68771	1.45410	.71373	1.40109	29
32	.63789	1.56767	.66272	1.50893	.68814	1.45320	.71417	1.40022	28
33	.63830	1.56667	.66314	1.50797	.68857	1.45229	.71461	1.39936	27
34	.63871	1.56566	.66356	1.50702	.68900	1.45139	.71505	1.39850	26
35	.63912	1.56466	.66398	1.50607	.68942	1.45049	.71549	1.39764	25
36	.63953	1.56366	.66440	1.50512	.68985	1.44958	.71593	1.39679	24
37	.63994	1.56265	.66482	1.50417	.69028	1.44868	.71637	1.39593	23
38	.64035	1.56165	.66524	1.50322	.69071	1.44778	.71681	1.39507	22
39	.64076	1.56065	.66566	1.50228	.69114	1.44688	.71725	1.39421	21
40	.64117	1.55966	.66608	1.50133	.69157	1.44598	.71769	1.39336	20
41	.64158	1.55866	.66650	1.50038	.69200	1.44508	.71813	1.39250	19
42	.64199	1.55766	.66692	1.49944	.69243	1.44418	.71857	1.39165	18
43	.64240	1.55666	.66734	1.49849	.69286	1.44329	.71901	1.39079	17
44	.64281	1.55567	.66776	1.49755	.69329	1.44239	.71946	1.38994	16
45	.64322	1.55467	.66818	1.49661	.69372	1.44149	.71990	1.38909	15
46	.64363	1.55368	.66860	1.49566	.69416	1.44060	.72034	1.38824	14
47	.64404	1.55269	.66902	1.49472	.69459	1.43970	.72078	1.38738	13
48	.64446	1.55170	.66944	1.49378	.69502	1.43881	.72122	1.38653	12
49	.64487	1.55071	.66986	1.49284	.69545	1.43792	.72167	1.38568	11
50	.64528	1.54972	.67028	1.49190	.69588	1.43703	.72211	1.38484	10
51	.64569	1.54873	.67071	1.49097	.69631	1.43614	.72255	1.38399	9
52	.64610	1.54774	.67113	1.49003	.69675	1.43525	.72299	1.38314	8
53	.64652	1.54675	.67155	1.48909	.69718	1.43436	.72344	1.38229	7
54	.64693	1.54576	.67197	1.48816	.69761	1.43347	.72388	1.38145	6
55	.64734	1.54478	.67239	1.48722	.69804	1.43258	.72432	1.38060	5
56	.64775	1.54379	.67282	1.48629	.69847	1.43169	.72477	1.37976	4
57	.64817	1.54281	.67324	1.48536	.69891	1.43080	.72521	1.37891	3
58	.64858	1.54183	.67366	1.48442	.69934	1.42992	.72565	1.37807	2
59	.64899	1.54085	.67409	1.48349	.69977	1.42903	.72610	1.37722	1
60	.64941	1.53986	.67451	1.48256	.70021	1.42815	.72654	1.37638	0
′	57° Cotang	57° Tang	56° Cotang	56° Tang	55° Cotang	55° Tang	54° Cotang	54° Tang	′

′	36° Tang	36° Cotang	37° Tang	37° Cotang	38° Tang	38° Cotang	39° Tang	39° Cotang	′
0	.72654	1.37638	.75355	1.32704	.78129	1.27994	.80978	1.23490	60
1	.72699	1.37554	.75401	1.32624	.78175	1.27917	.81027	1.23416	59
2	.72743	1.37470	.75447	1.32544	.78222	1.27841	.81075	1.23343	58
3	.72788	1.37386	.75492	1.32464	.78269	1.27764	.81123	1.23270	57
4	.72832	1.37302	.75538	1.32384	.78316	1.27688	.81171	1.23196	56
5	.72877	1.37218	.75584	1.32304	.78363	1.27611	.81220	1.23123	55
6	.72921	1.37134	.75629	1.32224	.78410	1.27535	.81268	1.23050	54
7	.72966	1.37050	.75675	1.32144	.78457	1.27458	.81316	1.22977	53
8	.73010	1.36967	.75721	1.32064	.78504	1.27382	.81364	1.22904	52
9	.73055	1.36883	.75767	1.31984	.78551	1.27306	.81413	1.22831	51
10	.73100	1.36800	.75812	1.31904	.78598	1.27230	.81461	1.22758	50
11	.73144	1.36716	.75858	1.31825	.78645	1.27153	.81510	1.22685	49
12	.73189	1.36633	.75904	1.31745	.78692	1.27077	.81558	1.22612	48
13	.73234	1.36549	.75950	1.31666	.78739	1.27001	.81606	1.22539	47
14	.73278	1.36466	.75996	1.31586	.78786	1.26925	.81655	1.22467	46
15	.73323	1.36383	.76042	1.31507	.78834	1.26849	.81703	1.22394	45
16	.73368	1.36300	.76088	1.31427	.78881	1.26774	.81752	1.22321	44
17	.73413	1.36217	.76134	1.31348	.78928	1.26698	.81800	1.22249	43
18	.73457	1.36134	.76180	1.31269	.78975	1.26622	.81849	1.22176	42
19	.73502	1.36051	.76226	1.31190	.79022	1.26546	.81898	1.22104	41
20	.73547	1.35968	.76272	1.31110	.79070	1.26471	.81946	1.22031	40
21	.73592	1.35885	.76318	1.31031	.79117	1.26395	.81995	1.21959	39
22	.73637	1.35802	.76364	1.30952	.79164	1.26319	.82044	1.21886	38
23	.73681	1.35719	.76410	1.30873	.79212	1.26244	.82092	1.21814	37
24	.73726	1.35637	.76456	1.30795	.79259	1.26169	.82141	1.21742	36
25	.73771	1.35554	.76502	1.30716	.79306	1.26093	.82190	1.21670	35
26	.73816	1.35472	.76548	1.30637	.79354	1.26018	.82238	1.21598	34
27	.73861	1.35389	.76594	1.30558	.79401	1.25943	.82287	1.21526	33
28	.73906	1.35307	.76640	1.30480	.79449	1.25867	.82336	1.21454	32
29	.73951	1.35224	.76686	1.30401	.79496	1.25792	.82385	1.21382	31
30	.73996	1.35142	.76733	1.30323	.79544	1.25717	.82434	1.21310	30
31	.74041	1.35060	.76779	1.30244	.79591	1.25642	.82483	1.21238	29
32	.74086	1.34978	.76825	1.30166	.79639	1.25567	.82531	1.21166	28
33	.74131	1.34896	.76871	1.30087	.79686	1.25492	.82580	1.21094	27
34	.74176	1.34814	.76918	1.30009	.79734	1.25417	.82629	1.21023	26
35	.74221	1.34732	.76964	1.29931	.79781	1.25343	.82678	1.20951	25
36	.74267	1.34650	.77010	1.29853	.79829	1.25268	.82727	1.20879	24
37	.74312	1.34568	.77057	1.29775	.79877	1.25193	.82776	1.20808	23
38	.74357	1.34487	.77103	1.29696	.79924	1.25118	.82825	1.20736	22
39	.74402	1.34405	.77149	1.29618	.79972	1.25044	.82874	1.20665	21
40	.74447	1.34323	.77196	1.29541	.80020	1.24969	.82923	1.20593	20
41	.74492	1.34242	.77242	1.29463	.80067	1.24895	.82972	1.20522	19
42	.74538	1.34160	.77289	1.29385	.80115	1.24820	.83022	1.20451	18
43	.74583	1.34079	.77335	1.29307	.80163	1.24746	.83071	1.20379	17
44	.74628	1.33998	.77382	1.29229	.80211	1.24672	.83120	1.20308	16
45	.74674	1.33916	.77428	1.29152	.80258	1.24597	.83169	1.20237	15
46	.74719	1.33835	.77475	1.29074	.80306	1.24523	.83218	1.20166	14
47	.74764	1.33754	.77521	1.28997	.80354	1.24449	.83268	1.20095	13
48	.74810	1.33673	.77568	1.28919	.80402	1.24375	.83317	1.20024	12
49	.74855	1.33592	.77615	1.28842	.80450	1.24301	.83366	1.19953	11
50	.74900	1.33511	.77661	1.28764	.80498	1.24227	.83415	1.19882	10
51	.74946	1.33430	.77708	1.28687	.80546	1.24153	.83465	1.19811	9
52	.74991	1.33349	.77754	1.28610	.80594	1.24079	.83514	1.19740	8
53	.75037	1.33268	.77801	1.28533	.80642	1.24005	.83564	1.19669	7
54	.75082	1.33187	.77848	1.28456	.80690	1.23931	.83613	1.19599	6
55	.75128	1.33107	.77895	1.28379	.80738	1.23858	.83662	1.19528	5
56	.75173	1.33026	.77941	1.28302	.80786	1.23784	.83712	1.19457	4
57	.75219	1.32946	.77988	1.28225	.80834	1.23710	.83761	1.19387	3
58	.75264	1.32865	.78035	1.28148	.80882	1.23637	.83811	1.19316	2
59	.75310	1.32785	.78082	1.28071	.80930	1.23563	.83860	1.19246	1
60	.75355	1.32704	.78129	1.27994	.80978	1.23490	.83910	1.19175	0
′	53° Cotang	53° Tang	52° Cotang	52° Tang	51° Cotang	51° Tang	50° Cotang	50° Tang	′

′	40°		41°		42°		43°		′
	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	
0	.83910	1.19175	.86929	1.15037	.90040	1.11061	.93252	1.07237	60
1	.83960	1.19105	.86980	1.14969	.90093	1.10996	.93306	1.07174	59
2	.84009	1.19035	.87031	1.14902	.90146	1.10931	.93360	1.07112	58
3	.84059	1.18964	.87082	1.14834	.90199	1.10867	.93415	1.07049	57
4	.84108	1.18894	.87133	1.14767	.90251	1.10802	.93469	1.06987	56
5	.84158	1.18824	.87184	1.14699	.90304	1.10737	.93524	1.06925	55
6	.84208	1.18754	.87236	1.14632	.90357	1.10672	.93578	1.06862	54
7	.84258	1.18684	.87287	1.14565	.90410	1.10607	.93633	1.06800	53
8	.84307	1.18614	.87338	1.14498	.90463	1.10543	.93688	1.06738	52
9	.84357	1.18544	.87389	1.14430	.90516	1.10478	.93742	1.06676	51
10	.84407	1.18474	.87441	1.14363	.90569	1.10414	.93797	1.06613	50
11	.84457	1.18404	.87492	1.14296	.90621	1.10349	.93852	1.06551	49
12	.84507	1.18334	.87543	1.14229	.90674	1.10285	.93906	1.06489	48
13	.84556	1.18264	.87595	1.14162	.90727	1.10220	.93961	1.06427	47
14	.84606	1.18194	.87646	1.14095	.90781	1.10156	.94016	1.06365	46
15	.84656	1.18125	.87698	1.14028	.90834	1.10091	.94071	1.06303	45
16	.84706	1.18055	.87749	1.13961	.90887	1.10027	.94125	1.06241	44
17	.84756	1.17986	.87801	1.13894	.90940	1.09963	.94180	1.06179	43
18	.84806	1.17916	.87852	1.13828	.90993	1.09899	.94235	1.06117	42
19	.84856	1.17846	.87904	1.13761	.91046	1.09834	.94290	1.06056	41
20	.84906	1.17777	.87955	1.13694	.91099	1.09770	.94345	1.05994	40
21	.84956	1.17708	.88007	1.13627	.91153	1.09706	.94400	1.05932	39
22	.85006	1.17638	.88059	1.13561	.91206	1.09642	.94455	1.05870	38
23	.85057	1.17569	.88110	1.13494	.91259	1.09578	.94510	1.05809	37
24	.85107	1.17500	.88162	1.13428	.91313	1.09514	.94565	1.05747	36
25	.85157	1.17430	.88214	1.13361	.91366	1.09450	.94620	1.05685	35
26	.85207	1.17361	.88265	1.13295	.91419	1.09386	.94676	1.05624	34
27	.85257	1.17292	.88317	1.13228	.91473	1.09322	.94731	1.05562	33
28	.85308	1.17223	.88369	1.13162	.91526	1.09258	.94786	1.05501	32
29	.85358	1.17154	.88421	1.13096	.91580	1.09195	.94841	1.05439	31
30	.85408	1.17085	.88473	1.13029	.91633	1.09131	.94896	1.05378	30
31	.85458	1.17016	.88524	1.12963	.91687	1.09067	.94952	1.05317	29
32	.85509	1.16947	.88576	1.12897	.91740	1.09003	.95007	1.05255	28
33	.85559	1.16878	.88628	1.12831	.91794	1.08940	.95062	1.05194	27
34	.85609	1.16809	.88680	1.12765	.91847	1.08876	.95118	1.05133	26
35	.85660	1.16741	.88732	1.12699	.91901	1.08813	.95173	1.05072	25
36	.85710	1.16672	.88784	1.12633	.91955	1.08749	.95229	1.05010	24
37	.85761	1.16603	.88836	1.12657	.92008	1.08686	.95284	1.04949	23
38	.85811	1.16535	.88888	1.12501	.92062	1.08622	.95340	1.04888	22
39	.85862	1.16466	.88940	1.12435	.92116	1.08559	.95395	1.04827	21
40	.85912	1.16398	.88992	1.12369	.92170	1.08496	.95451	1.04766	20
41	.85963	1.16329	.89045	1.12303	.92224	1.08432	.95506	1.04705	19
42	.86014	1.16261	.89097	1.12238	.92277	1.08369	.95562	1.04644	18
43	.86064	1.16192	.89149	1.12172	.92331	1.08306	.95618	1.04583	17
44	.86115	1.16124	.89201	1.12106	.92385	1.08243	.95673	1.04522	16
45	.86166	1.16056	.89253	1.12041	.92439	1.08179	.95729	1.04461	15
46	.86216	1.15987	.89306	1.11975	.92493	1.08116	.95785	1.04401	14
47	.86267	1.15919	.89358	1.11909	.92547	1.08053	.95841	1.04340	13
48	.86318	1.15851	.89410	1.11844	.92601	1.07990	.95897	1.04279	12
49	.86368	1.15783	.89463	1.11778	.92655	1.07927	.95952	1.04218	11
50	.86419	1.15715	.89515	1.11713	.92709	1.07864	.96008	1.04158	10
51	.86470	1.15647	.89567	1.11648	.92763	1.07801	.96064	1.04097	9
52	.86521	1.15579	.89620	1.11582	.92817	1.07738	.96120	1.04036	8
53	.86572	1.15511	.89672	1.11517	.92872	1.07676	.96176	1.03976	7
54	.86623	1.15443	.89725	1.11452	.92926	1.07613	.96232	1.03915	6
55	.86674	1.15375	.89777	1.11387	.92980	1.07550	.96288	1.03855	5
56	.86725	1.15308	.89830	1.11321	.93034	1.07487	.96344	1.03794	4
57	.86776	1.15240	.89883	1.11256	.93088	1.07425	.96400	1.03734	3
58	.86827	1.15172	.89935	1.11191	.93143	1.07362	.96457	1.03674	2
59	.86878	1.15104	.89988	1.11126	.93197	1.07299	.96513	1.03613	1
60	.86929	1.15037	.90040	1.11061	.93252	1.07237	.96569	1.03553	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	49°		48°		47°		46°		

′	44° Tang	44° Cotang	′	′	44° Tang	44° Cotang	′	′	44° Tang	44° Cotang	′
0	.96569	1.03553	60	20	.97700	1.02355	40	40	.98843	1.01170	20
1	.96625	1.03493	59	21	.97756	1.02295	39	41	.98901	1.01112	19
2	.96681	1.03433	58	22	.97813	1.02236	38	42	.98958	1.01053	18
3	.96738	1.03372	57	23	.97870	1.02176	37	43	.99016	1.00994	17
4	.96794	1.03312	56	24	.97927	1.02117	36	44	.99073	1.00935	16
5	.96850	1.03252	55	25	.97984	1.02057	35	45	.99131	1.00876	15
6	.96907	1.03192	54	26	.98041	1.01998	34	46	.99189	1.00818	14
7	.96963	1.03132	53	27	.98098	1.01939	33	47	.99247	1.00759	13
8	.97020	1.03072	52	28	.98155	1.01879	32	48	.99304	1.00701	12
9	.97076	1.03012	51	29	.98213	1.01820	31	49	.99362	1.00642	11
10	.97133	1.02952	50	30	.98270	1.01761	30	50	.99420	1.00583	10
11	.97189	1.02892	49	31	.98327	1.01702	29	51	.99478	1.00525	9
12	.97246	1.02832	48	32	.98384	1.01642	28	52	.99536	1.00467	8
13	.97302	1.02772	47	33	.98441	1.01583	27	53	.99594	1.00408	7
14	.97359	1.02713	46	34	.98499	1.01524	26	54	.99652	1.00350	6
15	.97416	1.02653	45	35	.98556	1.01465	25	55	.99710	1.00291	5
16	.97472	1.02593	44	36	.98613	1.01406	24	56	.99768	1.00233	4
17	.97529	1.02533	43	37	.98671	1.01347	23	57	.99826	1.00175	3
18	.97586	1.02474	42	38	.98728	1.01288	22	58	.99884	1.00116	2
19	.97643	1.02414	41	39	.98786	1.01229	21	59	.99942	1.00058	1
20	.97700	1.02355	40	40	.98843	1.01170	20	60	1.00000	1.00000	0
′	45° Cotang	45° Tang	′	′	45° Cotang	45° Tang	′	′	45° Cotang	45° Tang	′

NATURAL SECANTS AND COSECANTS.

Degrees.	Secants.							
	0′	10′	20′	30′	40′	50′	60′	
0	1.00000	1.00001	1.00002	1.00004	1.00007	1.00011	1.00015	89
1	1.00015	1.00021	1.00027	1.00034	1.00042	1.00051	1.00061	88
2	1.00061	1.00072	1.00083	1.00095	1.00108	1.00122	1.00137	87
3	1.00137	1.00153	1.00169	1.00187	1.00205	1.00224	1.00244	86
4	1.00244	1.00265	1.00287	1.00309	1.00333	1.00357	1.00382	85
5	1.00382	1.00408	1.00435	1.00463	1.00491	1.00521	1.00551	84
6	1.00551	1.00582	1.00614	1.00647	1.00681	1.00715	1.00751	83
7	1.00751	1.00787	1.00825	1.00863	1.00902	1.00942	1.00983	82
8	1.00983	1.01024	1.01067	1.01111	1.01155	1.01200	1.01247	81
9	1.01247	1.01294	1.01342	1.01391	1.01440	1.01491	1.01543	80
10	1.01543	1.01595	1.01649	1.01703	1.01758	1.01815	1.01872	79
11	1.01872	1.01930	1.01989	1.02049	1.02110	1.02171	1.02234	78
12	1.02234	1.02298	1.02362	1.02428	1.02494	1.02562	1.02630	77
13	1.02630	1.02700	1.02770	1.02842	1.02914	1.02987	1.03061	76
14	1.03061	1.03137	1.03213	1.03290	1.03368	1.03447	1.03528	75
15	1.03528	1.03609	1.03691	1.03774	1.03858	1.03944	1.04030	74
16	1.04030	1.04117	1.04206	1.04295	1.04385	1.04477	1.04569	73
17	1.04569	1.04663	1.04757	1.04853	1.04950	1.05047	1.05146	72
18	1.05146	1.05246	1.05347	1.05449	1.05552	1.05657	1.05762	71
19	1.05762	1.05869	1.05976	1.06085	1.06195	1.06306	1.06418	70
20	1.06418	1.06531	1.06645	1.06761	1.06878	1.06995	1.07115	69
21	1.07115	1.07235	1.07356	1.07479	1.07602	1.07727	1.07853	68
22	1.07853	1.07981	1.08109	1.08239	1.08370	1.08503	1.08636	67
23	1.08636	1.08771	1.08907	1.09044	1.09183	1.09323	1.09464	66
24	1.09464	1.09606	1.09750	1.09895	1.10041	1.10189	1.10338	65
25	1.10338	1.10488	1.10640	1.10793	1.10947	1.11103	1.11260	64
26	1.11260	1.11419	1.11579	1.11740	1.11903	1.12067	1.12233	63
27	1.12233	1.12400	1.12568	1.12738	1.12910	1.13083	1.13257	62
28	1.13257	1.13433	1.13610	1.13789	1.13970	1.14152	1.14335	61
29	1.14335	1.14521	1.14707	1.14896	1.15085	1.15277	1.15470	60
30	1.15470	1.15665	1.15861	1.16059	1.16259	1.16460	1.16663	59
31	1.16663	1.16868	1.17075	1.17283	1.17493	1.17704	1.17918	58
32	1.17918	1.18133	1.18350	1.18569	1.18790	1.19012	1.19236	57
33	1.19236	1.19463	1.19691	1.19920	1.20152	1.20386	1.20622	56
34	1.20622	1.20859	1.21099	1.21341	1.21584	1.21830	1.22077	55
35	1.22077	1.22327	1.22579	1.22833	1.23089	1.23347	1.23607	54
36	1.23607	1.23869	1.24134	1.24400	1.24669	1.24940	1.25214	53
37	1.25214	1.25489	1.25767	1.26047	1.26330	1.26615	1.26902	52
38	1.26902	1.27191	1.27483	1.27778	1.28075	1.28374	1.28676	51
39	1.28676	1.28980	1.29287	1.29597	1.29909	1.30223	1.30541	50
40	1.30541	1.30861	1.31183	1.31509	1.31837	1.32168	1.32501	49
41	1.32501	1.32838	1.33177	1.33519	1.33864	1.34212	1.34563	48
42	1.34563	1.34917	1.35274	1.35634	1.35997	1.36363	1.36733	47
43	1.36733	1.37105	1.37481	1.37860	1.38242	1.38628	1.39016	46
44	1.39016	1.39409	1.39804	1.40203	1.40606	1.41012	1.41421	45
	60′	50′	40′	30′	20′	10′	0′	Degrees.
	Cosecants.							

NATURAL SECANTS AND COSECANTS—(*Continued*).

Degrees.	Cosecants.							
	0′	10′	20′	30′	40′	50′	60′	
0	∞	343.77516	171.88831	114.59301	85.94561	68.75736	57.29869	89
1	57.29869	49.11406	42.97571	38.20155	34.38232	31.25758	28.65371	88
2	28.65371	26.45051	24.56212	22.92559	21.49368	20.23028	19.10732	87
3	19.10732	18.10262	17.19843	16.38041	15.63679	14.95788	14.33559	86
4	14.33559	13.76312	13.23472	12.74550	12.29125	11.86837	11.47371	85
5	11.47371	11.10455	10.75849	10.43343	10.12752	9.83912	9.56677	84
6	9.56677	9.30917	9.06515	8.83367	8.61379	8.40466	8.20551	83
7	8.20551	8.01565	7.83443	7.66130	7.49571	7.33719	7.18530	82
8	7.18530	7.03962	6.89979	6.76547	6.63633	6.51208	6.39245	81
9	6.39245	6.27719	6.16607	6.05886	5.95536	5.85539	5.75877	80
10	5.75877	5.66533	5.57493	5.48740	5.40263	5.32049	5.24084	79
11	5.24084	5.16359	5.08863	5.01585	4.94517	4.87649	4.80973	78
12	4.80973	4.74482	4.68167	4.62023	4.56041	4.50216	4.44541	77
13	4.44541	4.39012	4.33622	4.28366	4.23239	4.18238	4.13357	76
14	4.13357	4.08591	4.03938	3.99393	3.94952	3.90613	3.86370	75
15	3.86370	3.82223	3.78166	3.74198	3.70315	3.66515	3.62796	74
16	3.62796	3.59154	3.55587	3.52094	3.48671	3.45317	3.42030	73
17	3.42 30	3.38808	3.35649	3.32551	3.29512	3.26531	3.23607	72
18	3.23607	3.20737	3.17920	3.15155	3.12440	3.09774	3.07155	71
19	3.07155	3.04584	3.02057	2.99574	2.97135	2.94737	2.9238	70
20	2.92380	2.90063	2.87785	2.85545	2.83342	2.81175	2.79043	69
21	2.79043	2.76945	2.74881	2.72850	2.70851	2.68884	2.66947	68
22	2.66947	2.65040	2.63162	2.61313	2.59491	2.57698	2.55930	67
23	2.55930	2.54190	2.52474	2.50784	2.49119	2.47477	2.45859	66
24	2.45859	2.44264	2.42692	2.41142	2.39614	2.38107	2.36620	65
25	2.36620	2.35154	2.33708	2.32282	2.30875	2.29487	2.28117	64
26	2.28117	2.26766	2.25432	2.24116	2.22817	2.21535	2.20269	63
27	2.20269	2.19019	2.17786	2.16568	2.15366	2.14178	2.13005	62
28	2.13005	2.11847	2.10704	2.09574	2.08458	2.07356	2.06267	61
29	2.06267	2.05191	2.04128	2.03077	2.02039	2.01014	2.0 000	60
30	2.00000	1.98998	1.98008	1.97029	1.96062	1.95106	1.94160	59
31	1.94160	1.93226	1.92302	1.91388	1.90485	1.89591	1.88708	58
32	1.88708	1.87834	1.86990	1.86116	1.85271	1.84435	1.83608	57
33	1.83608	1.82790	1.81981	1.81180	1.80388	1.79604	1.78829	56
34	1.78829	1.78062	1.77303	1.76552	1.75808	1.75073	1.74345	55
35	1.74345	1.73624	1.72911	1.72205	1.71506	1.70815	1.70130	54
36	1.70130	1.69452	1.68782	1.68117	1.67460	1.66809	1.66164	53
37	1.66164	1.65526	1.64894	1.64268	1.63648	1.63035	1.62427	52
38	1.62427	1.61825	1.61229	1.60639	1.60054	1.59475	1.58902	51
39	1.58902	1.58333	1.57771	1.57213	1.56661	1.56114	1.55572	50
40	1.55572	1.55036	1.54504	1.53977	1.53455	1.52938	1.52425	49
41	1.52425	1.51918	1.51415	1.50916	1.50422	1.49933	1.49448	48
42	1.49448	1.48967	1.48491	1.48019	1.47551	1.47087	1.46628	47
43	1.46628	1.46173	1.45721	1.45274	1.44831	1.44391	1.43956	46
44	1.43956	1.43524	1.43096	1.42672	1.42251	1.41835	1.41421	45
	60′	50′	40′	30′	20′	10′	0′	
	Secants.							Degrees.

PART II.

STRENGTH OF MATERIALS, AND STABILITY OF STRUCTURES.

INTRODUCTION.

In the following chapters the author has endeavored to give the necessary rules, formulas, and data for computing the strength and stability of all ordinary forms of building construction, whether of wood, steel, or masonry, and in fact of all but the more intricate problems of steel construction, with which few architects care to cope, and which, indeed, are more especially within the province of the trained engineer.

The rules and formulas have been reduced to their simplest form or expression, and, in general, require only an elementary knowledge of mathematics to understand them. Every pains has also been taken to show the application of the formulas and to preserve their accuracy, and it is believed that they represent the most intelligent practice of the present day.

In giving constants for the strength of materials, the author has been guided by the practice of leading structural engineers, by the available records of tests, and by his own experience of many years as a practising and consulting architect. The varying conditions of building construction have also been taken into account, and an attempt made to adapt the values to the practical conditions usually governing such construction. Every possible precaution has been taken to prevent the misapplication of rules and formulas and to insure absolute safety without undue waste of materials.

Much thought and labor has been given to the preparation of the numerous tables contained in this portion of the book, both to insure their accuracy and to arrange them in the most convenient shape for instant use by architects and builders. Nearly all of these tables were computed by the author, and all have been carefully verified, and it is believed that they may be used with perfect confidence. In all cases they give the same values that would be obtained by using the formulas, while affording a great saving of time and labor, as well as obviating any chance for error in making the necessary computations.

Owing to the nature of the book, and the large number of pages required to compass its scope as a book of reference, some forms of construction, such as foundations, masonry and

fireproof construction, roof trusses, etc., have necessarily been treated rather briefly, and more in the way of giving necessary data than of going into an elaborate description or discussion of the principles involved. Those persons who wish a more complete treatise on foundations and masonry in general, the author would refer to Part I. of his work on *Building Construction and Superintendence*, which supplements the rules and data herein contained. References to various works containing more complete information on some subjects are also made in the different chapters.

EXPLANATION OF SIGNS AND TERMS USED IN THE FOLLOWING FORMULAS.

Besides the usual arithmetical signs and characters in general use, the following characters and abbreviations will frequently be used:

The sign $\sqrt{}$ means square root of number behind.

$\sqrt[3]{}$ means cube root of number behind.

( ) means that all the numbers between are to be taken as one quantity.

. means decimal parts; $2.5=2\frac{5}{10}$, or $.46=\frac{46}{100}$.

′ denotes feet.

″ denotes inches.

The letter A denotes the co-efficient of strength for beams one inch square, and one foot between the supports.

C denotes resistance, in pounds, of a block of any material to crushing, per square inch of section.

E denotes the modulus of elasticity of any material, in pounds per square inch.

e denotes constant for stiffness of beams.

F denotes resistance of any material to shearing, per square inch.

R denotes the modulus of rupture of any material.

S denotes a factor of safety.

T denotes resistance of any material to being pulled apart, in pounds, per square inch of cross-section.

$\times$ between letters or words, denotes multiplication.

[Note. In a few places in the book it has been necessary to give a different meaning to one or more of the above letters but in all such cases the new meaning has been very clearly indicated.]

Breadth is used to denote the horizontal thickness of a beam or the least side of a rectangular post or strut, and is always measured in inches.

Depth denotes the vertical height of a beam or girder, and is always to be taken in inches, unless expressly stated otherwise.

Length denotes the distance *between supports* and *in feet*, unless otherwise specified.

Abbreviations.—In order to shorten the formulas, it has often been found necessary to use certain abbreviations, such as bet. for between, bot. for bottom, dist. for distance, diam. for diameter, hor. for horizontal, sq. for square, etc., which, however, can in no case lead to uncertainty as to their meaning.

Where the word "ton" is used in this volume, it always means 2000 pounds.

CHAPTER I.

DEFINITIONS OF TERMS USED IN MECHANICS.

THE following terms frequently occur in treating of mechanical construction, and it is essential that their meaning be well understood.

Mechanics is the science which treats of the action of forces.

Applied Mechanics treats of the laws of mechanics which relate to works of human art; such as beams, trusses, arches, etc.

Rest is the relation between two points, when the straight line joining them does not change in length or direction.

A body is at rest relatively to a point, when any point in the body is at rest relatively to the first-mentioned point.

Motion is the relation between two points, when the straight line joining them changes in length or direction, or in both.

A body moves relatively to a point, when any point in the body moves relatively to the point first mentioned.

Force is that which changes, or tends to change, the state of a body in reference to rest or motion. It is a cause regarding the essential nature of which we are ignorant. We cannot deal with forces properly, but only with the laws of their action.

Equilibrium is that condition of a body in which the forces acting upon it balance or neutralize each other.

Statics is that part of Applied Mechanics which treats of the conditions of equilibrium, and is divided into:—

a. Statics of rigid bodies.

b. Hydrostatics.

In building we have to deal only with the former.

Structures are artificial constructions in which all the parts are intended to be in equilibrium and at rest, as in the case of a bridge or roof-truss.

They consist of two or more solid bodies, called **pieces,** which are connected at portions of their surfaces called **joints.**

There are three conditions of equilibrium in a structure, viz.:—

I. The forces exerted on the whole structure must balance each other. These forces are:—

a. The weight of the structure.

b. The load it carries.

c. The supporting pressures, or resistance of the foundations, called external forces.

II. The forces exerted on each piece must balance each other. These forces are:—

a. The weight of the piece.

b. The load it carries.

c. The resistance of its joints.

III. The forces exerted on each of the parts into which any piece may be supposed to be divided must balance each other.

Stability consists in the fulfilment of conditions I. and II., that is, the ability of the structure to resist displacement of its parts.

Strength consists in the fulfilment of condition III., that is, the ability of a piece to resist breaking.

Stiffness consists in the ability of a piece to resist bending.

The theory of structures is divided into two parts; viz.:—

I. That which treats of strength and stiffness, dealing only with single pieces, and generally known as **strength of materials.**

II. That which treats of stability, dealing with structures.

Stress.—The load or system of forces acting on any piece of material is often denoted by the term "stress," and the word will be so used in the following pages.

The *intensity of the stress* per square inch on any normal surface of a solid is the total stress divided by the area of the section in square inches. Thus, if we had a bar ten feet long and two inches square, with a load of 8000 pounds pulling in the direction of its length, the stress on any normal section of the rod would be 8000 pounds; and the intensity of the stress per square inch would be $8000 \div 4$, or 2000 pounds.

Strain.—When a solid body is subjected to any kind of stress, an alteration is produced in the volume and figure of the body, and this alteration is called the "strain." In the case of the bar given above, the strain would be the amount that the bar would stretch under its load.

The **Ultimate Strength,** or **Breaking Load,** of a body is the load required to produce fracture in some specified way.

The **Safe Load** is the load that a piece can support without impairing its strength.

Factors of Safety.—When not otherwise specified, a *factor of safety* means the ratio in which the breaking load exceeds the safe load. In designing a piece of material to sustain a certain load, it is required that it shall be perfectly safe under all circumstances; and hence it is necessary to make an allowance for any defects in the material, workmanship, etc. It is obvious, that, for materials of different composition, different factors of safety will be required. Thus, steel being more homogeneous than wood, and less liable to defects, it does not require so great a factor of safety. And, again, different kinds of strains require different factors of safety. Thus, a long wooden column or strut requires a greater factor of safety than a wooden beam. As the factors thus vary for different kinds of strains and materials, we will give the proper factors of safety for the different strains when considering the resistance of the material to those strains.

Unit Stress is the allowed stress per unit of measurement, generally the square inch, and corresponds to intensity.

Distinction between Dead and Live Load.—The term "dead load," as used in mechanics, means a load that is applied by imperceptible degrees, and that remains steady; such as the weight of the structure itself.

A "live load" is one that is applied suddenly, or accompanied with vibrations; such as swift trains travelling over a railway-bridge, or a force exerted in a moving machine.

It has been found by experience that the effect of a live load on a beam or other piece of material is twice as severe as that of a dead load of the same weight: hence a piece of material designed to carry a live load should have a factor of safety *twice* as large as one designed to carry a dead load.

The load produced by a crowd of people walking on a floor is usually considered to produce an effect which is a mean between that of a dead and live load, and a factor of safety is adopted accordingly.

In municipal ordinances and laws relating to the load on floors, the load to be supported by the floor, exclusive of its inherent construction, and of stationary fixtures, is generally referred to as the "live load," no matter of what it may consist; but the term does not have the significance given to it by engineers, and as defined in the paragraph above.

The **Modulus of Rupture** is a constant quantity found

in the formulas for the strength of beams, and is eighteen times the value of the constant "A" used for wooden beams.

In recent works the term *fibre stress* is more frequently used, and represents the same quantity.

Modulus of Elasticity.—If we take a bar of any elastic material, one inch square, and of any length, secured at one end, and to the other apply a force pulling in the direction of its length, we shall find by careful measurement that the bar has been stretched or elongated by the action of the force.

Now, if we divide the total elongation in inches by the original length of the bar in inches, we shall have the elongation of the bar per unit of length; and, if we divide the pulling force per square inch by this latter quantity, we shall have what is known as the modulus of elasticity.

Hence we may define the *modulus of elasticity as the pulling or compressing force per unit of section divided by the elongation or compression per unit of length.*

As an example of the method of determining the modulus of elasticity of any material we will take the following:—

Suppose we have a bar of wrought iron, two inches square and ten feet long, securely fastened at one end, and to the other end we apply a pulling-force of 40,000 pounds. This force causes the bar to stretch, and by careful measurement we find the elongation to be 0.0414 of an inch. Now, as the bar is ten feet, or 120 inches, long, if we divide 0.0414 by 120, we shall have the elongation of the bar per unit of length.

Performing this operation, we have as the result 0.00034 of an inch. As the bar is two inches square, the area of cross-section is four square inches, and hence the pulling-force per square inch is 10,000 pounds. Then, dividing 10,000 by 0.00034, we have as the modulus of elasticity of the bar 29,400,000 pounds.

This is the method generally employed to determine the modulus of elasticity of iron ties; but it can also be obtained from the deflection of beams, and it is in that way that the values of the modulus for most woods have been found.

Another definition of the modulus of elasticity, and which is a natural consequence of the one just given, is the number of pounds that would be required to stretch or shorten a bar one inch square by an amount equal to its length, provided that the law of perfect elasticity would hold good for so great a range

The modulus of elasticity is generally denoted by E, and is used in the determination of the stiffness of beams.

Moment.—If we take any solid body, and pivot it at any point, and apply a force to the body, acting in any direction except in a line with the pivot, we shall produce rotation of the body, provided the force is sufficiently strong. This rotation is produced by what is called the *moment* of the force; and *the moment of a force* about any given point or pivot is the product of the force into the perpendicular distance from the pivot to the line of action of the force, or, in common phrase, *the product of the force into the arm with which it acts.*

The Centre of Gravity of a body is the point through which the resultant of the weight of the body always acts, no matter in what position the body be. If a body be suspended at its centre of gravity, and revolved in any direction, it will always be in equilibrium.

(For centre of gravity of surfaces, lines, and solids, see Chap. V.)

CLASSIFICATION OF STRAINS WHICH MAY BE PRODUCED IN A SOLID BODY.

The different strains to which building materials may be exposed are:—

I. **Tension,** as in the case of a weight suspended from one end of a rod, rope, tie-bar, etc., the other end being fixed, tending to stretch or lengthen the fibres.

II. **Shearing Strain,** as in the case of rivets, treenails, pins in bridges, etc., where equal forces are applied on opposite sides in such a manner as to tend to force one part over the adjacent one.

III. **Compression,** as in the case of a weight resting on top of a column or post, tending to compress the fibres.

IV. **Transverse** or **Cross Strain,** as in the case of a load on a beam, tending to bend it.

V. **Torsion,** a twisting strain, which seldom occurs in building construction, though quite frequently in machinery.

Combined Strains.—The parts of structures are often subjected to two or more of the above strains at the same time, as in the case of "strut beams" and "tie beams," and all beams and girders are subjected to a shearing strain, as well as to a transverse strain.

CHAPTER II.

FOUNDATIONS AND SPREAD FOOTINGS.

The term "foundation" is used to designate all that portion of any structure which serves only as a basis on which to erect the superstructure.

This term is sometimes applied to that portion of the solid material of the earth upon which the structure rests, and also to the artificial arrangements which may be made to support the base.

In the following pages these will be designated by the term "foundation-bed."

Object of Foundations.—The object to be obtained in the construction of any foundation is to form such a solid base for the superstructure that no movement shall take place after its erection. But all structures built of coarse masonry, whether of stone, or brick, will settle to a certain extent; and, with a few exceptions, all soils will become compressed under the weight of almost any building.

The main object of the architect or engineer, therefore, is not to prevent settlement entirely, but to insure that it shall be uniform; so that, after the structure is finished, it will have no cracks or flaws, however irregularly it may be disposed over the area of its site.

Nature and Bearing Power of Soils.—The architect should in all cases endeavor to discover the nature of the soil upon which the building is to be built before commencing the foundation plans, as a foundation that will prove satisfactory in one soil or locality may not be sufficient for another.

For most buildings a sufficient idea of the soil may be obtained from an inspection of adjoining excavations,* or from inquiry amongst builders who have erected buildings on adjoining lots.

Many soils, however, vary greatly within a comparatively small area. This is especially true of soils composed of strata of different materials, as sand or gravel and clay, and very often

these strata have a decided dip, so that they are encountered at different levels under different portions of the building. For these reasons, therefore, the character and bearing power of the soil under all large or heavy buildings should be determined at different points by borings or excavations, unless the composition of the soil is homogenous and fully known.

Testing Soils.— For ordinary buildings borings to the depth of 6 feet below the bottom of the trenches will be sufficient to determine the composition of the soil. These should preferably be made by a 6-inch auger, but a 4-inch auger may be used if a larger one cannot be had, and the borings examined for every foot in depth and memoranda made of the same. For very heavy and costly buildings the bearing power of the soil, even when apparently of firm earth, is often determined by testing. Clay soils, especially, vary much in their bearing capacity, and are most frequently tested. Good sand or gravel will seldom need to be tested.

Tests may be made with a platform resting on four legs, or by a large pole or mast. The test should be made in several places, and always at the proposed depth of the footings.

The ground under the Congressional Library at Washington, D.C., was tested by means of a traveling car having four cast-iron pedestals, each one foot square at the base and set 4 feet apart each way. In testing the soil under the State Capitol at Albany, N. Y., the load was placed on a mast 12 inches square, held vertically by guys, with a cross-frame to hold the weights. The bottom of the mast was set in a hole 3 feet deep, 18 inches square at the top, and 14 at the bottom.* A permanent bench mark should be established before loading, and accurate levels taken by means of an engineer's level before the load is applied, and frequent levels taken as the load is gradually increased until a sinkage is shown. From one-fifth to one-half of the load required to produce settlement is generally adopted for the safe load according to circumstances.

Values for the Bearing Power of Soils. — The following values for the bearing power of soils, given by Prof. Ira O. Baker,† of the University of Illinois, have been quite generally accepted by engineers:

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* For more complete descriptions of these tests see "Building Construction and Superintendence," Part I, p. 20.

† See "Treatise on Masonry Construction," p. 194.

TABLE I.—BEARING POWER OF SOILS.

Kind of material.	Bearing power in tons per square foot. Min.	Max.
Rock—the hardest—in thick layers, in native bed	200	—
Rock equal to best ashlar masonry	25	30
Rock equal to best brick masonry	15	20
Rock equal to poor brick masonry	5	10
Clay on thick beds, always dry	4	6
Clay on thick beds, moderately dry	2	4
Clay, soft	1	2
Gravel and coarse sand, well cemented	8	10
Sand, compact and well cemented	4	6
Sand, clean, dry	2	4
Quicksand, alluvial soils, etc.	0.5	1

When deciding upon the pressure which may safely be put upon the soil several practical considerations should be taken into account. "For example, the pressure on the foundation of a tall chimney should be considerably less than that of the low massive foundation of a fire-proof vault. In the former case a slight inequality of bearing power, and consequent unequal settling, might endanger the stability of the structure; while in the latter no serious harm would result. The pressure per unit of area should be less for a light structure subject to the passage of heavy loads—as for example a railroad viaduct—than for a heavy structure, subject only to a quiescent load, since the shock and jar of the moving load are far more serious than the heavier quiescent load." *

The pressure under piers supporting a tier of columns should also be a little greater than under a masonry wall, so that the pier may settle a little more to allow for the compression in the joints of the mason-work of the wall. Usually an increase of pressure of about 10 per cent. may be allowed.

The following example of the known weight on different soils will give a very good idea of what has been done in actual practice.

Rock.—St. Rollox chimney, poorest kind of sandstone, 2 tons per sq. ft.

Clay.—Chimney, McCormick Reaper Works, Chicago, $1\frac{3}{4}$ tons per square foot on dry, hard clay.

Capitol at Albany, N. Y., rests on blue clay containing from 60 to 90 per cent. of alumina, the remainder being fine sand,

* Ira O. Baker, "American Architect," November 3, 1888.

and containing 40 per cent. of water on an average. The safe load was taken at 2 tons per square foot.

In the case of the Congressional Library at Washington, which rests on "yellow clay mixed with sand," 2½ tons per square foot was taken for the safe load. "Experience in Central Illinois shows that if the foundation is carried down below the action of the frost the clay subsoil will bear 1½ to 2 tons per square foot without appreciable settling." *

Sand and Gravel.—"In an experiment in France clean river sand compacted in a trench supported 100 tons per square foot.

"The piers of the Cincinnati Suspension Bridge are founded on a bed of coarse gravel 12 feet below water; the maximum pressure on the gravel is 4 tons per square foot.

"The piers of the Brooklyn Suspension Bridge are founded 44 feet below the bed of the river; upon a layer of sand 2 feet thick resting upon bed-rock; the maximum pressure is about 5½ tons per square foot.

"At Chicago sand and gravel about 15 feet below the surface are successfully loaded with 2 to 2½ tons per square foot.

"At Berlin the safe load for sandy soil is generally taken at 2 to 2½ tons per square foot.

"The Washington Monument, Washington, D. C., rests upon a bed of very fine sand 2 feet thick. The ordinary pressure on certain parts of the foundation being not far from 11 tons per square foot, which the wind may increase to nearly 14 tons per square foot." *

The Home Insurance Building, La Salle and Adams St., Chicago, was proportioned for a bottom pressure of 2 tons per square foot. Settled 2¼ inches.

"Probably none of the high buildings on spread footings settled less than 6 inches. The amount of settlement generally is between 6 and 12 inches. The Auditorium settled more than 20 inches under the tower." †

Bearing Power of Soils, as Fixed by Municipal Laws.— Many of the larger cities prescribe the maximum pressure to be placed on the ground under the footings, although as a rule the laws are somewhat indefinite as regards the nature of the soil.

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* Ira O. Baker, "American Architect," November 3, 1888.

† E. C. Shankland in "Inland Architect," January, 1898.

The building code of Greater New York specifies the following as the maximum permissible loads for different soils:

"Soft clay, one ton per square foot;

"Ordinary clay and sand together, in layers, wet and springy, two tons per square foot;

"Loam, clay, or fine sand, firm and dry, three tons per square foot;

"Very firm coarse sand, stiff gravel, or hard clay, four tons per square foot, or as otherwise determined by the Commissioner of Buildings having jurisdiction."

The requirements of the Chicago Building Ordinance is as follows:

"If the soil is a layer of pure clay at least fifteen feet thick without admixture of any foreign substance excepting gravel, it shall not be loaded more than at the rate of 3,500 pounds per square foot. If the soil is a layer of pure clay at least fifteen feet thick and is dry and thoroughly compressed, it may be loaded not to exceed 4,500 pounds per square foot.

"If the soil is a layer of dry sand fifteen feet or more in thickness, and without admixture of clay, loam, or other foreign substance, it shall not be loaded more than at the rate of 4,000 pounds per square foot.

"If the soil is a mixture of clay and sand, it shall not be loaded more than at the rate of 3,000 pounds per square foot."

Proportioning the Footings.—The footings under dwellings and light buildings, when on firm soil, are usually proportioned according to the thickness of the wall above, rather than by the pressure on the soil, as the weight of such buildings, when not more than three stories high, will seldom exceed $1\frac{3}{4}$ tons per square foot when distributed by the footings. The width of the footings, however, even in light buildings, should be proportioned so that the pressure on the soil will be approximately the same per square foot under all parts of the building. It is owing largely to the unequal pressure on the soil, as where wide openings occur, or where one wall is higher than the adjacent, that cracks occur in brick and stone walls.

For high and heavily loaded buildings, the area of the footings should be carefully proportioned both to the load and to the bearing power of the soil.

In computing the weight to be supported by the footings, all of the dead or permanent load, such as the weight of the materials entering into and forming a part of the building should be

taken, and to this should be added only so much of the live or movable load that the floors are to support as will probably be upon the floor most of the time, as to secure uniform settlement, it is necessary that the loads for which the footings are proportioned shall be as near the actual weight of the building as possible.

For warehouses, stores, etc., about 50 per cent. of the live load for which the floor beams are proportioned should be added to the dead load supported on the footings.

For office-buildings, hotels, etc., the weight of the people who may occupy them should be neglected altogether in proportioning the footings, and only about 15 lbs. per square foot of floor allowed to cover the weight of furniture, books, safes, etc.

For theatres and similar buildings some allowance should probably be made for the weight of the people, the actual amount depending upon the arrangement of the plan and the character of the soil. Almost any soil, after it has been compacted by the dead weight of the building, will carry a shifting load of people without further settlement, while if the footings were computed to carry the full live loads for which the floor beams were designed, it would be found when the building was finished that the actual loads on the footings under the walls would be much greater than under the columns, and if the ground had settled at all during the erection of the building, the probabilities would be that the building would be higher in the centre than at the walls.*

Municipal Requirements as to Proportioning Footings to Live Loads.—The Building Code of Greater New York requires that footings shall be proportioned as follows:

"The load exerting pressure under the footings of foundations in buildings more than three stories in height are to be computed as follows: For warehouses and factories they are to be the full dead load and the full live load established by this code. In stores and buildings for light manufacturing purposes, they are to be full dead load and 75 per cent. of the live

* It is the judgment of the best engineers that the areas of foundations on compressible soil should be proportioned to the dead loads only, and not to theoretical or occasional loads. When live loads have been figured on both the interior columns and on the columns in the exterior walls, the exterior columns have always been found to settle more, from the fact that the live load forms a larger percentage of the interior column loads than of the wall column loads.—J. K. Freitag.

load established by this code. The same applies to churches, schoolhouses, and places of public assembly. In office-buildings, hotels, dwellings, apartment houses, tenement houses, lodging houses, and stables, they are to be the full dead load and 60 per cent. of the live load established by this code." The footings must be designed to distribute the loads as uniformly as possible, so as not to exceed the safe bearing capacity of the soil as established in this code."

The Chicago ordinance merely specifies that "Foundations shall be proportioned to the actual average loads they will have to carry, and not to theoretical and occasional loads."

The Boston Building Laws make no specific requirements as to how the loads shall be computed.

The following examples illustrate the proper method of proportioning the area of footings:

Example 1.—To proportion the footings under a six-story warehouse, with solid side walls of brick, and iron columns and steel girders, spaced 16 ft. c. to c. across the building, and the same distance longitudinally, the safe bearing capacity of the soil being assumed at 3 tons per square foot.

Computation—Walls.

Cubic feet of brickwork in one lineal foot of side wall, from footing to top of fire-wall 164; at 120 lbs. per ft.	= 19,680 lbs.
Floor area supported by 1 ft. of bearing wall, 8 ft. in each story.	
Actual weight of materials in 1 sq. ft. of floor, 75 lbs. 75×8 ft.×6 floors	= 3,600 lbs.
Actual weight of 8 sq. ft. of roof, 60×8	= 480 lbs.
Probable constant load on first three floors, 50 lbs. per sq. ft., 50×8×3	= 1,200 lbs.
Probable constant load on 4th, 5th, and 6th floors, 40 lbs. per sq. ft., 8×40×3	= 960 lbs.
Probable constant load on roof	——
Total load on 1 lineal foot of footing	=25,920 lbs.

25,920÷6,000 (3 tons) = 4 ft. 4 ins. width of footing.

Under Columns.

Weight of one tier of columns from footing to roof, including fireproof covering and plaster	=12,000 lbs.
Floor area in each story supported by column 16×16 feet = 256 sq. ft.	

Load under columns from preceding page 12,000 lbs.	
Actual weight of 256 sq. ft. of floor for 6 stories $(256 \times 75 \times 6)$	=115,200 lbs.
Actual weight of 256 sq. ft. of roof, at 60 lbs.	= 15,360 lbs.
Probable constant load on first three floors, 50 lbs. $\times 256 \times 3$	= 38,400 lbs.
Probable constant load on 4th, 5th, and 6th floors, 40 lbs. $\times 256 \times 3$	= 30,720 lbs.
Probable constant load on roof	——
Total load on footings	=211,680 lbs.

$211{,}680 \div 6{,}600$ (3 tons increased 10%) = 32 sq. ft. = area of footing.

The front and rear walls would probably be divided into piers by large openings and should be proportioned in the same way as the column footings, except that only the piers supporting ends of girders would be figured for floor loads.

Only warehouses for storage of heavy merchandise should be figured for a probable constant load of 50 lbs. For ordinary merchandise 30 lbs. would be more nearly correct.*

For an office-building or hotel, the calculation would be the same for the dead load, but the probable constant load would not exceed 15 lbs.

Example 2.—To proportion the footings under the tower, front and side walls of a church, built on ground capable of sustaining 4 tons to the square foot.

Data.—Tower walls, 82 ft. high above footings; 18 ft. of wall 2 ft. thick, 13 ft. of wall 20″ thick, 51 ft. of wall 16″ thick.

Side wall adjacent to tower, 36 ft. high, 14 ft. of wall 20″ thick, balance of wall 16″ thick.

The front wall is divided in the centre by a wide opening, leaving piers 12 ft. wide at each side one pier being adjacent to the tower.

Computation.—It is proposed to make the footings under the side wall 3 ft. wide and to proportion the other footings to the same unit pressure.

Weight of masonry in one lineal foot of side wall	6,320 lbs.
Weight of floor material and pews supported by one lineal foot of side wall	200 lbs.

* The floor beams, of course, should be computed for the full *possible* load.

Weight of roof and ceiling supported by one lineal foot of side wall.............................. 200 lbs.
Weight of snow on roof, and people on floors disregarded.

Total weight on one lineal foot of footing.. 6,720 lbs.

6,720÷3 ft. (width of footing)=2,240 lbs. per sq. ft. on trenches, which should be used as a basis for proportioning all other footings.

Weight of masonry in one lineal foot of tower wall.. 15,080 lbs.
Weight of floors supported by one lineal foot of tower wall.. 296 lbs.
Weight of roof supported by one lineal foot of tower wall.. 140 lbs.

Total weight of one lineal foot of footing... 15,516 lbs.

15,516÷2,240=7 ft.=width of footing.

Each of the 12-ft. piers of front wall contains and supports masonry weighing.................. 149,040 lbs.
Weight of gallery and pews supported by each pier. 3,876 lbs.

Total weight on 12 ft. of footing.................. 152,916 lbs. or 12,743 lbs. per lineal ft.

12,743÷2,240 lbs.=5.1 ft. width of footing.

As the pressure on the soil in this case is so slight, the width of the tower footings could be reduced to 6 ft. and of the front-wall footings to 4′—4″ without causing cracks where the walls join, but the theoretical width should always be computed.

Where the unit pressure approaches closely to the safe bearing of the soil no reduction should be made from the computed widths.

Centre of Pressure Should Coincide with Centre of Base.—That the walls and piers of a building may settle uniformly and without producing cracks in the superstructure it is not only essential that the area of the footings shall be in proportion to the load and the bearing power of the soil, but also that the *centre of pressure* (a vertical line through the centre of gravity of the wall or pier) *shall pass through the centre of the footing.*

This condition is of the first importance, for if the centre of pressure does not coincide with the centre of the footing, or base, the ground will yield most on the side which is nearest to the centre of pressure, and, as the ground yields, the base assumes an inclined position, often tilting the lower portion of

the wall or pier, and producing unsightly cracks in the superstructure.

Foundations on Rock.—To prepare a rock foundation for being built upon, all that is generally required is to cut away the loose and decayed portions of the rock, and to dress the rock to a plane surface as nearly perpendicular to the direction of the pressure as is practicable; or, if the rock forms an inclined plane, to cut a series of plane surfaces, like those of steps, for the wall to rest on. If there are any fissures in the rock they should be filled with concrete. Concrete is better than masonry for this purpose, as, when once set, it is nearly incompressible under anything short of a crushing force; so that it forms a base almost as solid as the rock itself, while the compression of the mortar joints of masonry is certain to cause some irregular settlement.

If it is unavoidably necessary that some parts of the foundation shall start from a lower level than others, care should be taken to keep the mortar joints as close as possible, or to execute the lower portions of the work in cement, or some hard-setting mortar; otherwise the foundations will settle unequally and thus cause much injury to the superstructure. The load placed on the rock should at no time exceed one-eighth of that necessary to crush it. When building on a ledge much trouble is often caused by the water which collects on top of the stone, and stands or runs on its surface. Some method of draining the water is absolutely necessary if the basement is to be kept dry.

Foundations on Clay.—This soil is found in every condition, varying from slate or shale, which will support almost any load, to a soft, damp material, which will squeeze out in every direction when a moderately heavy pressure is brought upon it.

Ordinary clay soils, however, when they can be kept dry, will carry any usual load without trouble, but as a rule clay soils will give more trouble than either sand, gravel, or stone.

In the first place, the top of the footings must be carried below the frost-line to prevent heaving, and for the same reason the outside face of the wall should be built with a slight batter, about $\frac{5}{8}''$ to the foot, and *perfectly smooth.* The frost-line varies with different localities, attaining a depth of six feet in some of the very Northern States, although between three and four feet is the usual depth in the so-called Northern States. The effect

of freezing and thawing on clay soils is very much greater than on other soils.

The surface of the ground around the building should be graded so that the rain-water will run away from the building, and in most clays subsoil drains are necessary. When the clay occurs in inclined layers great care must be exercised to prevent it from sliding, and when building on a side hill the utmost precautions must be taken to exclude water from the soil, for if the clay becomes wet the pressure of the walls may cause it to ooze from under the footings. The erection of very heavy buildings in such locations must be considered hazardous, even when every precaution is taken.

Should it be necessary to carry a portion of the foundations to a greater depth than the rest, the lower portion of the walls should be built as described under "Foundations on Rock," and care must be taken to prevent the upper part of the bed from slipping. Wherever possible the footings should be carried all around the building at the same level.

If the clay contains coarse sand or gravel its supporting power is increased, and it is less liable to slide or ooze away.

Foundations on Sand and Gravel.—Gravel gives less trouble than any other material as a foundation bed. It does not settle under any ordinary load, and will safely carry the heaviest of buildings if the footings are properly proportioned. It is not affected by water, provided it is confined laterally, so that the sand and fine gravel cannot wash out. This soil is also not greatly affected by frost.

Sand also makes an excellent foundation bed when confined laterally, and is practically incompressible, as clean river sand compacted in a trench has been known to support 100 tons to the square foot.

As long as the sand is confined on all sides, and the footings are all on the same level, no trouble whatever will be encountered, unless it be in the caving of the banks in making the excavations. Should the cellar be excavated to different levels, however, sufficient retaining walls must be erected where the depth changes to prevent the sand of the upper level from being forced out from under the footings, and precautions should be taken in such a case to keep water from penetrating under the upper footings.

Foundations on Loam and Made Land. — No foundation should start on loam (soil containing vegetable

matter), or on land that has been made or filled in, unless, indeed, the filling consist of clean beach sand, which, when settled with water, may be considered equal to the natural soil.

Loam should always be penetrated to the firm soil beneath, and when the made land or filling overlies a firm earth, the footings should be carried to the natural soil. When the filled land is always wet, as on the coast or the borders of a lake, piles may be used, extending into the firm earth, and the tops cut off below low-water mark; but piles should never be used where it is not certain that they will be always wet.

Foundations for Chimneys.—As examples of the foundations required for very high chimneys we quote the following from a treatise on foundations, in the latter part of a work on "Foundations and Foundation Walls," by George T. Powell.

Fig. 1 represents the base of a chimney erected in 1859 for the Chicago Refining Company, 151 feet high, and 12 feet square at the foot. The base, merely two courses of heavy dimension

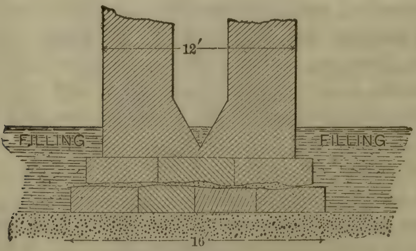

Fig. 1.

stone, as shown, is bedded upon the surface-gravel near the mouth of the river, there recently deposited by the lake. The mortar employed in the joint between the stone is roofing-gravel in cement. The area of the base is 256 square feet, the weight of chimney, inclusive of base, 625 tons, giving a pressure of 2.44 tons to the square foot. This foundation proved to be perfect.

Fig. 2 represents the base of a chimney erected in 1872 for the McCormick Reaper Works, Chicago, which is 160 feet high, 14 feet square at the foot, with a round flue of 6 feet 8 inches diameter.

The base covers 625 square feet; the weight of the chimney and base is approximately 1,100 tons; the pressure upon the ground, (dry hard clay) is therefore 1.76 tons to the square foot. This foundation also proved to be perfect in every respect.

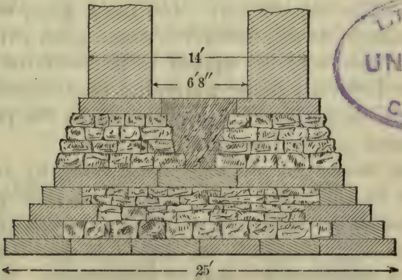

Fig. 2.

Pile Foundations.—When it is required to build upon a compressible soil that is *constantly saturated* with water and of considerable depth, the most practicable method of obtaining a solid and enduring foundation for buildings of moderate height is by driving piles.

A large proportion of the buildings in the city of Boston, Mass., and several of the tall office-buildings of New York City and Chicago, rest on piles, and piles are extensively used for supporting buildings, grain elevators, etc., erected along the water front of coast and lake cities.

The durability of piles in ground constantly saturated with water is beyond question, as several instances exist where piles have been found in a perfectly sound condition after the lapse of from six to seventeen centuries.

Municipal Requirements.—The laws of Boston require that piles shall be capped with granite, and that the spacing shall not exceed 3 ft. between centres. The laws of Chicago require that piles shall be driven to rock or hard pan and be capped with an oak grillage; they also specify a maximum load of 25 tons per pile and a maximum fibre stress of 1,200 pounds per square inch for the oak grillage.

The laws of New York specify a minimum diameter of 5 inches, a maximum spacing of 3 feet between centres, and a maximum safe load of 20 tons per pile.

The Piles are made from the trunks of trees; they should be as straight as possible and not less than 5 ins. in diameter at small end for light buildings, or 8 ins. for heavy buildings. The woods generally used for piles in the Northern States are the spruce, hemlock, white pine, Norway pine, Georgia pine, and occasionally oak, hickory, elm, black gum, and basswood. In the Southern States, Georgia or pitch pine, cypress and oak are used. There does not appear to be much difference in the woods as to durability under water, but the tougher and stronger woods are to be preferred, especially where the piles are to be driven to hard pan and heavily loaded.

The piles should be prepared for driving by cutting off all limbs close to the trunk, and sawing the ends square. It is probably .better to remove the bark, although piles are more often driven with the bark on, and it is doubtful if the bark makes much difference one way or the other.

For driving in soft and silty soils, experience has shown that the piles drive better with a square point. When the penetration is less than 6 ins. at each blow the top of the pile should be protected from "brooming" by putting on an iron ring about 1 inch less in diameter than the head of the pile, and from $2\frac{1}{2}$ to 3 inches wide by $\frac{5}{8}''$ thick. The head should be chamfered to fit the ring. When driven into compact soil, such as sand, gravel or stiff clay the point of the pile should be shod with iron or steel. The method shown at *A*, Fig. 3, answers very well for all but very hard soils, and for these a cast conical point about 5 inches in diameter, secured by a long dowel, with a ring around the end of the pile, as shown at *B*, makes the best shoe.

Piles that are to be driven in or exposed to salt water should be thoroughly impregnated with creosote, dead oil or coal tar, or some mineral poison to protect them from the "teredo" or ship worm, which will completely honeycomb an ordinary pile in three or four years.

Driving.—The piles should always be driven to an even bearing, which is determined by the penetration under the last four or five blows of the hammer.

The usual method of driving piles for the support of buildings is by a successsion of blows given with a block of cast iron, or steel, called the "hammer," which slides up and down between the uprights of a machine called a "pile-driver." The machine is placed over the pile, so that the hammer descends fairly on its head, the piles always being driven with the small end

down. The hammer is generally raised by steam power, and is dropped either automatically or by hand. The usual weight of the hammers used for driving piles for building foundations is from 1,500 to 2,500 pounds, and the fall varies from 5 to 20 feet, the last blows being given with a short fall. Occasionally, hammers weighing up to 4,000 pounds and over are used.

Steam hammers are to a considerable extent taking the place of the ordinary drop-hammer in large cities, as they will drive many more piles in a day, and with less damage to the piles. The steam hammer delivers short quick blows, from 60 to 70

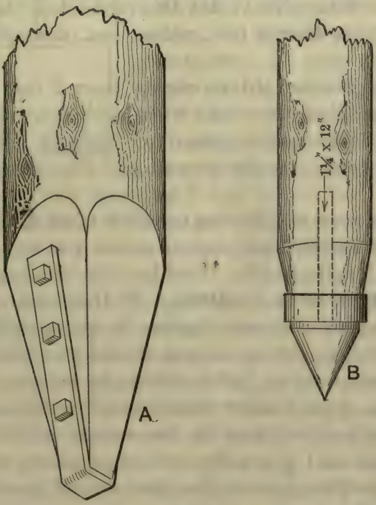

Fig. 3.

to the minute, and seems to jar the piles down, the short interval between the blows not giving time for the soil to settle around the pile.*

In driving piles care should be taken to keep them plumb, and when the penetration becomes small, the fall should be

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* The 5,000 piles, averaging 48 ft. in net length, under the new Chicago Post Office were driven with a steam hammer, weighing 4,400 lbs. and making 60 blows per minute.

reduced to about 5 feet, the blows being given in rapid succession.

Whenever a pile refuses to sink under several blows, before reaching the average depth, it should be cut off and another pile driven beside it.

When several piles have been driven to a depth of 20 feet or more and refuse to sink more than ½ inch under five blows of a 1,200-pound hammer falling 15 feet, it is useless to try them further, as the additional blows only result in brooming and crushing the head and point of the pile, and splitting and crushing the intermediate portions to an unknown extent.

Spacing.—Piles should not be spaced less than 2 feet on centres, nor more than 3 feet, unless iron or wooden grillage is used.

When long piles are driven nearer than 2 feet from centres there is danger that they may force each other up from their solid bed on the bearing stratum. Driving the piles close together also breaks up the ground and diminishes the bearing power.

When three rows of piles are used the most satisfactory spacing is 2 feet 6 inches from centres across the trench and 3 feet from centres longitudinally, provided this number of piles will carry the weight of the building. If they will not, then the piles must be spaced closer together longitudinally, or another row of piles driven, but in no case should the piles be less than 2 feet apart from centres, unless driven by means of a water jet.

The number of piles under the different portions of the building should be proportioned to the weight which they are to support, so that each pile will receive very nearly the same load.

Capping.—The tops of the piles should invariably be cut off at or a little below low water-mark, otherwise they will soon commence to decay. They should then be capped, either with large stone blocks, concrete or timber or steel grillage.

Granite Capping.—In Boston it is obligatory to cap the piles with blocks of granite, which rest directly on the tops of the piles. If the stone does not fit the surface of the pile, or a pile is a little low, it is wedged up with oak or stone wedges. In capping with stone a section of the foundation should be laid out on the drawings showing the arrangement of the capping stones.

A single stone may rest on one, two, or three piles, but not on four, nor on three piles in a straight line, as it is practically impos-

sible to make the stone bear evenly. Fig. 4 shows the best arrangement of the capping for three rows of piles. Under dwellings and light buildings the piles are often driven in two rows, staggered, in which case each stone should rest on three piles. After the piles are capped, large footing-stones, extending in one piece across the wall, should be laid in cement mortar on the capping.

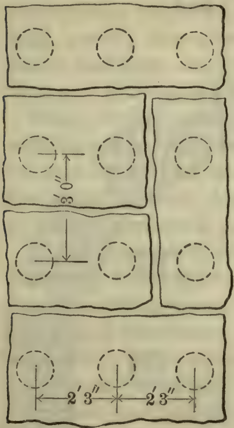

Fig. 4.

Fig. 5 shows a partial piling plan, with the arrangement of the cap stones, of the Boston Chamber of Commerce. It may seem that most of the stones rest on three piles, and a very few on two piles.

Concrete Capping.—In New York a very common method of capping is to excavate to a depth of 1 foot below the top of the piles and one foot outside of them, and fill the space thus excavated solid with Portland cement concrete, deposited in layers and well rammed.

After the concrete is brought level with the top of the piles additional layers are deposited over the whole width of the foundation until the concrete attains a depth of 18 inches above the piles. On this foundation brick or stone footings are laid as on solid earth. If long bars of twisted steel, about $\frac{3}{4}''$ square are imbedded in the concrete about 3 inches above the tops of the piles, this makes, in the opinion of the author, the best form of capping, the twisted bars giving great transverse strength to the concrete.

Grillage.—Most of the pile foundations of Chicago have heavy timber grillage bolted to the tops of the piles, and on these timbers are laid the stone or concrete footings.

The timbers for the grillage should be at least $10'' \times 10''$ in cross-section, and should have sufficient transverse strength to sustain the load from centre to centre of piles, using a low fibre stress. They should be laid longitudinally on top of the piles and be fastened to them by means of *drift bolts*, which are plain bars of iron, either round or square, driven into a hole about 20 per cent. smaller than the iron. One-inch round or square bars are generally used, the hole being bored by a $\frac{3}{4}$-inch

auger for the round bolts or a $\frac{7}{8}$-inch auger for the square bolts. The bolts should enter the pile at least 1 foot.

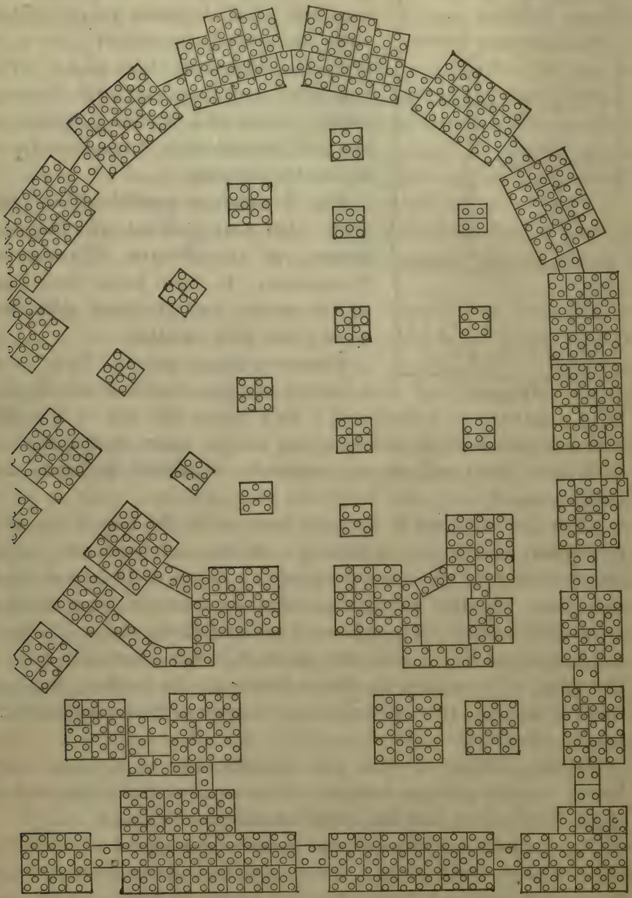

Fig. 5.

If heavy stone or concrete footings are used, and the space between the piles and timbers is filled with concrete level with

the top of the timbers, no more timbering is required; but if the footings are to be made of small stones, and no concrete is used, a solid floor of cross timbers, at least 6 inches thick, for heavy buildings, should be laid on top of the longitudinal capping and drift-bolted to them.

Where timber grillage is used it should, of course, be kept entirely below the lowest recorded water line, otherwise it will rot and allow the building to settle. It has been proved conclusively, however, that any kind of sound timber will last practically forever if completely immersed in water.

The advantages of timber grillage are that it is easily laid and effectually holds the tops of the piles in place. It also tends to distribute the pressure evenly over the piles, as the transverse strength of the timber will help to carry the load over a single pile, which for some reason may not have the same bearing capacity as the others. Steel beams, imbedded in concrete, are sometimes used to distribute the weight over piles, but some other form of construction can generally be employed at less expense and with equally good results.*

For **Concrete Piles,** see page 177.

Specifications for Pile Foundations.—This contractor is to furnish and drive the piles indicated on sheet No. 1.

The piles are to be of sound *spruce* (hemlock, Georgia pine) perfectly straight from end to end, trimmed close, and cut off square to the axis at both ends.

They must be not less than six (6) inches in diameter, at the small end, ten (10) inches at the large end, when cut off, and of sufficient length to reach solid bottom, the necessary length of piles to be determined by driving test piles in different parts of the foundation.

All piles to be driven vertically, in the exact positions shown by the plan, until they do not move more than five (5) inches under the last five blows of a hammer weighing 2,000 lbs. and falling twenty (20) feet. All split or shattered piles are to be removed if possible and a good one driven in place of each imperfect one. In cases where such piles cannot be removed an additional one must be driven for each imperfect one. If the piles show a tendency to broom, they shall be bound with a wrought-iron ring, 2½ ins. wide, and ½ in. thick.

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* For description of the pile foundations and capping of the Chicago Post Office, see Freitag's "Architectural Engineering," pp. 350–352.

All piles, when driven to the required depth shall be sawed off square and horizontal at the grade indicated on the drawings.

Bearing Power of Piles.—As used for supporting buildings, piles may be divided into two classes: A. Those which are driven to rock or "hard pan," i.e., firm gravel or clay, and (B) those which do not reach hard pan.

Piles of Class A, when driven through a soil that is sufficiently firm to brace the pile at every point, may be computed to sustain a load equal to the safe resistance to crushing on the least cross section. If the surrounding soil is plastic the bearing power of the pile will be its safe load computed as a column, having a length equal to the length of the pile when capped.

Test piles driven on the site of the Chicago Public Library through 27 ft. of soft, plastic clay, 23 ft. of tough compact clay, and 2 ft. into hard pan sustained a load of 50.7 tons per pile for two weeks without apparent settlement. There are many instances where piles driven to the depth of 20 ft. in hard clay sustain from 20 to 40 tons, and a few instances up to 80 tons per pile.

Piles of Class B depend for their bearing power upon the friction, cohesion, and buoyancy of the soil into which they are driven. The safe load for such piles is usually determined by the average penetration of the pile under the last four or five blows of the hammer. Several engineers have formulated rules for determining the safe load of piles of this class, but there are so many elements that modify the penetration, or its exact determination, as well as varying conditions in driving, and in the soil, that it is regarded an impossibility to formulate any rule that can be considered entirely satisfactory for all the conditions under which such piles are driven.

The formula now most generally used by engineers was derived by Mr. M. A. Wellington, and is often referred to as the *Engineering News* formula.

The formula is

$$\begin{matrix}\text{Safe load} \\ \text{in tons}\end{matrix} = \frac{2.\,w.\,h}{S+1},$$

in which w = weight of hammer in tons, h = height of fall of hammer in feet, S = penetration under last blow, or the average under the last five blows. When loads are based on this formula the piles should be driven until the penetration does not exceed the limit assumed, or if this is found to be impracticable, new calculations must be made based on the smallest average penetration that

can be obtained, and a greater number of piles used. In localities where piling is commonly used for obtaining the foundation, the least penetration that can be obtained within practical limits of length of pile can generally be ascertained by observation, or by consulting an experienced pile-driver. The longer the pile the less will be the final set or penetration as a rule. Where there is no experience to guide one it will be necessary to drive a few piles to determine the length of pile required, or the least set for a given length of pile. Some piles will have to be driven further than others to bring to an equal bearing. When the piles are to be loaded to more than 50 per cent. of the assumed safe load, the final set of each pile should be carefully measured by an inspector, the broom and splinters being removed from the head of the pile for the last blow.

The following table, computed by the above formula, gives the safe loads for different penetrations, under different falls, of a hammer weighing one ton. *For a hammer of different weight* multiply the safe load in table by the actual weight of hammer, in tons. Thus for a hammer weighing 1,000 lbs. the values in the table should be multiplied by ½ or for a 1,500 lb. hammer by ¾.

TABLE II.—SAFE LOADS, IN TONS, FOR PILES.

(Hammer weighing 1 ton.)

Penetration of Pile in inches.	Height of the fall of the hammer, in feet.												
	3	4	5	6	8	10	12	14	16	18	20	25	30
0.25	4.8	6.4	8.1	9.7	12.9	16.1	19.4	22.5	25.8	29.1	32.3		
0.50	4.0	5.3	6.7	8.0	10.7	13.3	16.1	18.7	21.3	24.0	26.6	33.3	
0.75	3.4	4.6	5.7	6.9	9.2	11.5	13.8	16.1	18.4	20.7	23.0	28.8	34.5
1.00	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	25.0	30.0
1.25		3.6	4.5	5.4	7.1	8.9	10.7	12.5	14.3	16.1	17.9	22.3	26.7
1.50		3.2	4.0	4.8	6.4	8.0	9.6	11.2	12.8	14.4	16.0	20.0	24.0
1.75			3.6	4.4	5.8	7.3	8.8	10.2	11.7	13.1	14.6	18.2	21.9
2.00			3.3	4.0	5.3	6.7	8.0	9.3	10.7	12.0	13.3	16.7	20.0
2.50				3.4	4.6	5.7	6.9	8.0	9.1	10.3	11.4	14.3	17.1
3.00				3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	12.5	15.0
3.50					3.6	4.4	5.3	6.2	7.1	8.0	8.9	11.1	13.3
4.00					3.2	4.0	4.8	5.6	6.4	7.2	8.0	10.0	12.0
5.00						3.3	4.0	4.7	5.3	6.0	6.7	8.3	10.0
6.00							3.4	4.0	4.6	5.1	5.7	7.1	8.6

Example of Pile Foundations.—As an example of the method of determining the necessary number of piles to support a given building, we will determine the number of piles required to support the side walls, and interior piers of the warehouse

computed in Example 1. The method of computing the weights being exactly the same in both cases, and the remarks regarding the weight of people being applicable to pile foundations as well as to foundations placed directly on the soil.

Data.—From observation of the pile-driving for an adjacent building it is found that piles driven from 20 to 30 feet, take a set of 1 inch under a 1,200-lb. hammer falling 20 feet, and that additional blows give about the same set.

Computation.—From the above table we find that the safe load for a fall of 20 ft. and penetration of 1 inch is 20 tons. Multiplying by the weight of our hammer in tons (.6), we have 12 tons as the safe load per pile.

Referring to the computations on page 139, we see that the total load on one lineal foot of footing is 25,920 lbs., or about 13 tons. As we must have at least two rows of piles, and the two piles will support 24 tons, it follows that the spacing of the piles longitudinally should be $24 \div 13 = 1$ ft. 10 ins. As this is too close, we should use 3 rows of piles, spaced 2 ft. apart, and the longitudinal spacing would then be $36 \div 13 = 2$ ft. 9 ins. The width of the capping would be about five feet.

Under the Interior Piers.—The load on the piles, under the interior columns (p. 140) is 211,680 lbs., or 105.8 tons. This divided by 12, the safe load for one pile, gives 9 piles, or three rows of three piles each, which should be spaced 2′ 6″ apart each way.

Some Instances of the Actual Load on Piles.—The following instances of the actual loads supported by piles, under well-known buildings, and of loads which piles have borne for a short time without settlement, should be of value when designing pile foundations.

Boston.—At the new Southern R.R. Station three piles were loaded with about 60 tons of pig iron (20 tons per pile), without settlement. The allowed load was 10 tons per pile.

Piles 12 ins. in diameter at the butt, 6 ins. at the point, driven 31 ft. in hard blue clay, near Haymarket Square, failed to show movement under 30 tons. Ultimate load believed to be 60 tons.* Other piles driven 17.9 ft. sustained a load of 31 tons. Average penetration under last ten blows of a 1,710-lb. hammer falling from 9 to 12 feet varied from 0.4 to 0.95 ins. per blow for fifteen piles.

Piles 25 ft. long under the Chamber of Commerce building

* Horace J. Howe, "American Architect," June 11, 1898.

penetrated about 3 ins. under the last blow of a 2,000-lb. ram falling about 15 feet.

Chicago.—New Public Library building;—piles proportioned to 30 tons each. Tested to 50.7 tons without settlement.

Schiller Building;—estimated load 55 tons per pile; building settled from 1½ to 2¼ ins.

Passenger Station, Northern Pacific Railroad, Harrison St.; piles 50 ft. long carry 25 tons each without perceptible settlement.

The Art Institute and portions of the Stock Exchange rest on piles, and also a large proportion of the warehouses and other buildings on the banks of the river.

New York City.—The Ivins (Park Row) Building is supported by about 3,500 fourteen-inch spruce piles, arranged in clusters of fifty or sixty, for single columns, and a corresponding number under piers supporting two or more columns.

The piles were driven to a refusal of 1 inch under a 20-foot blow of a 2,000-lb. hammer. Material, fine dense sand to a depth of over 90 feet. But few piles could be driven more than 15 or 20 feet. Average maximum load per pile, 9 tons.*

The American Tract Society Building is also supported by piles.

Brooklyn, N. Y.—Piles under the Government Graving Dock driven 32 ft. on the average in fine sand mixed with fine mica and a little vegetable loam are supposed to sustain from 10 to 15 tons each.

New Orleans.—Piles driven from 25 to 40 ft. in a soft alluvial soil carry safely from 15 to 25 tons with a factor of safety of 6 to 8.—Patton.

Cost of Driving Piles.—The cost of driving piles naturally varies with the character of the soil, and the conditions under which they are driven.

In New York City a 2,500-lb. drop-hammer drove 4 piles per day of 10 hours. With a steam-hammer, 13 piles per day were driven, for the same foundation. Piles were 70 ft. long, 8 ins. in diameter at the point, and 15 ins. at the head.

Average cost of driving 800 piles with the steam-hammer, $2 each.

In New York Harbor 1,800 piles were driven by a steam-hammer, 24 to 26 ft. into gravel and hard pan at a cost of 80 cents each.

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* For description of this foundation, see *Engineering Record* of July 23, 1898.

In Chicago, 40 Norway pine piles were driven 45 ft. deep every ten hours at a cost (for driving) of 55 cts. each. Another firm drove from 60 to 65 piles, each 45 ft. long, 15 ft. deep into hard sand each day at a cost of about 30 cts. each. In both cases steam-hammers were used.*

In Boston, spruce piles from 30 to 45 ft. long cost from $3 to $5 in place. Georgia pine piles as long as 70 ft. cost about $15 apiece for the piles themselves, and $2 or more each for the driving. Oak piles from 40 to 50 ft. long cost from $8 to $10 each in place.†

References.—A very valuable paper on "Some Instances of Piles and Pile-driving, New and Old," by Horace J. Howe, C.E., was published in the *American Architect and Building News*, commencing June 11, 1898. The paper records a great many tests and gives several formulas and many experiences of distinguished engineers.

Part I. of Building Construction and Superintendence also gives much additional information in regard to pile foundations and experiments on the bearing power of piles.

Much valuable information on piles is also given in "A Practical Treatise on Foundations," by W. M. Patton, C.E. John Wiley & Sons, publishers.

SPREAD FOUNDATIONS.

Compressible soils are often met with that will bear from 1 to 2 tons per square foot with very little settlement, and with a uniform settlement under the same unit pressure.

In such cases it is often cheaper to *spread* the foundations or footings so as to reduce the unit pressure to the capacity of the soil than to attempt to drive piles, or to go down to hard pan. Spread footings may be built of concrete with iron tension bars, of steel beams or girders, and concrete, or of timber.

Concrete with Iron Tension Bars.—When the necessary height can be obtained, spread footings composed of Portland cement concrete, with iron tension members, have many qualities to recommend them. Such footings are easy of construction, they are cheap, and their durability is everlasting. The iron being so completely imbedded in the concrete, it cannot

* *American Architect*, June 4, 1898, p. 78.

† George B. Francis, C.E., in *American Architect*, July 23, 1898.

rust,* and hence there is no possibility of deterioration in the footings.

By the use of twisted iron or other forms of tension rods the transverse strength of concrete footings may be made equal to that of steel beams, but concrete footings require more height.†

Fig. 6 shows the most economical section for a concrete and twisted iron footing. In building the footings with steel beams, the strength of the concrete is practically wasted, while in this method of construction it is all utilized. It has been proved that the entire tensile strength of the twisted bars can be utilized, and, being held continuously along their entire length by the concrete as a screw bolt is held by the nut, they neither draw nor stretch, except as the concrete extends with them.

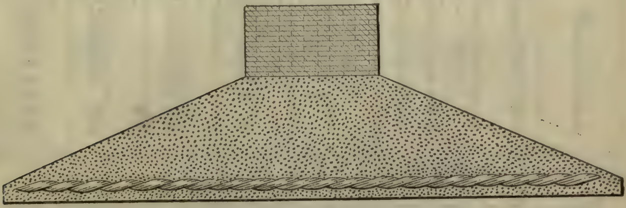

Fig. 6.

In building concrete footings, as shown in Fig. 6, a layer of concrete from 3 to 6 inches thick, made in the proportion of 1 to 3 should first be laid, and the iron bars laid on and tamped down into it. Another layer of 4 inches, mixed in the same proportion, should then be laid, after which the concrete may be mixed in the proportion of one to six. Each layer should be laid before the preceding layer has had time to harden, otherwise they may not adhere thoroughly.

The author has prepared Table III., giving the strength and proportions of footings constructed in this way, which he believes to have a large margin of safety. Two sizes of bars are

* In cutting through a portion of a foundation built of concrete and iron, and submerged in salt water, ten years after the work was done, no deterioration to the iron whatever was found. Iron imbedded in concrete, with the end projecting, has been found bright and clean after the projecting end had completely rusted away.

† For description of tension bars see Chap. XXIII.

given, with the corresponding safe loads for the footings, the other measurements applying to both cases. The measurements in the third column refer to the width of the brick or stone footing resting on the concrete. The greater the width of this footing in proportion to the width of the concrete, the less will be the strain on the tension rods.

TABLE III.—PROPORTIONS AND STRENGTH OF CONCRETE FOOTINGS WITH TWISTED IRON TENSION BARS.

Width of Footing in feet.	Thickness of concrete.		Width of Stone footing.		Distance between centres of bars.	Size of square bar.	Safe load per lineal foot.	Size of square bar.	Safe load per lineal foot.
	Ft.	In.	Ft.	In.	Inches.	Inches.	Tons.	Inches.	Tons.
20	3	6	6	0	8	2	78	1⅞	66
18	3	3	5	6	8	2	76	1¾	56
16	2	10	5	0	7	1¾	73	1½	50
14	2	8	4	8	7	1⅝	70	1⅜	49
12	2	6	4	4	6	1⅜	65	1¼	48
10	2	3	4	0	6	1¼	65	1	42
8	2	0	4	0	6	1	60	¾	40
6	1	8	3	6	6	¾	55	½	29

Piers.—Footings for piers may be built in the same manner, with two sets of bars laid crossways of each other, and also diagonally, as shown in Fig. 7. In the case of piers the pressure will be more evenly distributed if the corners are cut off. The same size of bars should be used for a pier as for a wall, whose footings have the same projection beyond the masonry, and the depth of the concrete should be the same.

Fig. 7 represents the construction of the pier footings under the interior columns of a four-story factory for the Pacific Coast Borax Co. at Bayonne, N. J. The footings are computed to resist an upward pressure of the ground of 2,500 lbs. per square foot.*

This form of construction has been used to a considerable extent in San Francisco and in the eastern States, and twisted iron in connection with concrete is being more extensively used every year. The right to use twisted iron in concrete is protected by letters patent, now owned by the Ransome Concrete

* For a description of this building and other illustrations, see *Engineering Record* of July 30, 1898.

Machinery Co. of New York.* The corrugated bars controlled by the St. Louis Expanded Metal Fireproofing Co. are also being extensively used for spread footings. For detailed information see 1903 catalogue of this company.

Steel Beam Footings.—When it is necessary to make the foundations from 8 to 15 feet wide, with a very small height to the footings, as is the case in Chicago, steel beams must be used to furnish the necessary transverse strength. Even when build-

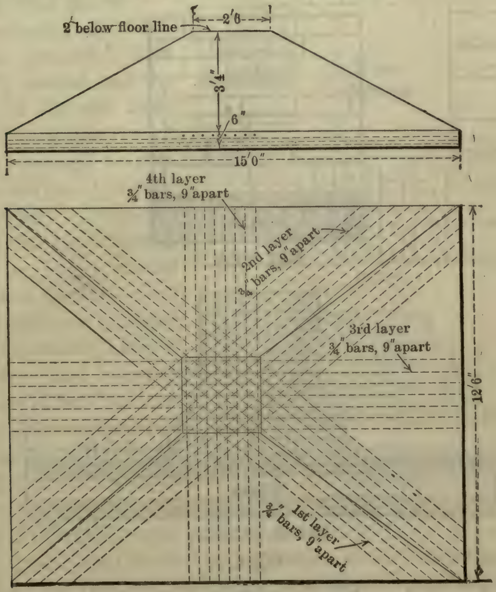

Fig. 7.

ing on solid ground, it is claimed that iron and steel footings for tall buildings, at the present price of steel, are cheaper than masonry footings. The author doubts, however, if steel footings will prove as durable as those of masonry.

When used under walls, the beams are laid in one course at right angles to the wall, and from 9 to 20 inches apart according to the size of the beams, thickness of the concrete and estimated

* Twisted bars purchased from this company may be used, however, without payment of further royalties.

pressure per square foot. Over the centre of the platform of beams is placed the brick or stone footings as shown in Fig. 8.

When used under piers, as is generally the case in the modern tall building, the beams are usually arranged in two layers, as shown in Fig. 9. The bottom layer contains a sufficient number of beams to cover the necessary bearing area. Above these is laid a second layer of beams, at right angles to the first, and long enough to reach the extreme outer edge of the outer beams of the first layer. Upon the centre of the upper tier of beams is placed

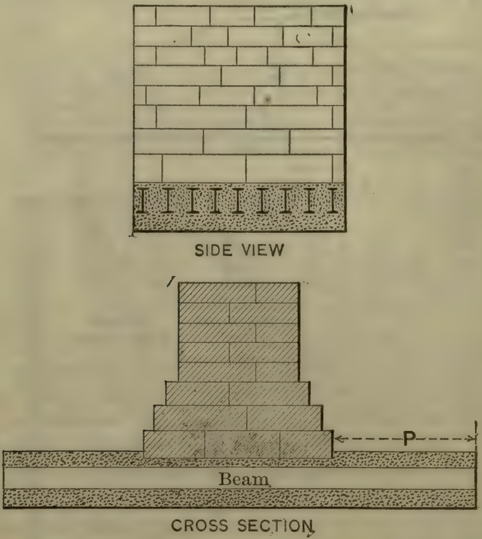

Fig. 8.

the iron shoe of the column, or a heavy stone base. Frequently the upper tier of beams is so wide that it cannot be well spanned by a shoe, in which case a third layer of beams or a short riveted girder or bolster is placed under the column.

When steel beam footings were first used, rails were employed for the beams on account of their lesser cost, and they were built up in five or six layers, but now that steel beams are so cheap it is much better to use I-beams for the grillage and to build the grillage in not more than two layers. When the upper layer is composed of several beams, the author believes that, owing to the bending of the beams in the lower layer, a greater strain is

brought on the two outer beams of the upper layer than on the beams between, and that it is impossible to determine the amount of this extra stress. For this reason the Author is strongly of the opinion that it is better engineering to use but two beams in the upper course of a pier footing, or if sufficiently large beams cannot be obtained, a single riveted girder.

In preparing the footings, the ground is first carefully leveled and the bottom of the pier located. If the ground is not compact enough to permit of excavating for the concrete bed without the sides of the pit or trench falling in, heavy planks or timbers should be set up and fastened together at the corners, and, if necessary, tied between with rods, to hold the concrete in place and prevent its spreading before it has thoroughly set. A layer of Portland cement concrete, made in the proportion of 1 to 6, and from 6 to 12 inches thick, according to the weight on the footings, should then be filled in between the timbers and well rammed and leveled off. If the concrete is to be 12 inches thick it should be deposited in two layers. Upon this concrete the beams should be carefully bedded in 1 to 2 Portland cement mortar, so as to bring them nearly level and in line with each other.

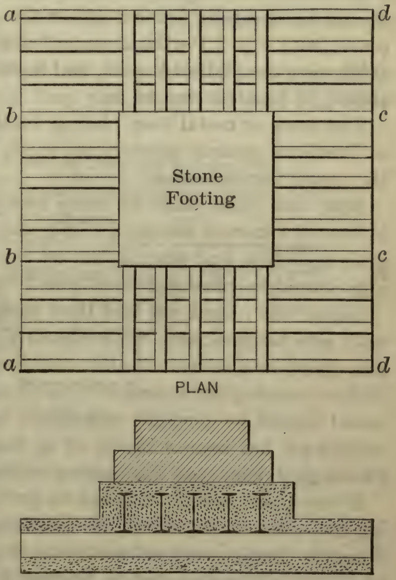

Fig. 9.

The distance apart of the beams, from centre to centre, must not be so great that the beams will crush through the concrete, and on the other hand there must be a space of at least 2 inches between the edges of flanges to permit the introduction of the concrete filling. As soon as the beams are in place the spaces between them should be filled with 1 to 6 concrete, the stone

being broken to pass through a $1\frac{1}{2}$-inch ring, and the concrete well rammed into place, so that no cavities will be left in the centre. The concrete must also be carried at least 3 inches beyond the beams on sides and ends, and kept in place by planks or timbers.

If two or more layers of beams are used, the top of each layer should be carefully leveled (after the concrete has been put in place) with 1 to 2 Portland cement mortar, not more than $\frac{1}{2}$ inch thick over the highest beams, and in this the next layer of beams should be bedded, and so on.

The stone or metal base plate or footing should also be bedded in Portland cement mortar, not more than $\frac{3}{4}$ inch thick, above the upper tier of beams.

After the base plate or stone footing is in place at least 3 inches of concrete should be laid above the beams and at the sides and ends, and when this is set the whole outside of the footings should be plastered with 1 to 2 Portland cement mortar.

Before the beams are laid they should be thoroughly cleaned with wire brushes, and, while absolutely dry, either painted with iron paint or else heated and coated with two coats of asphalt. Before covering the beams with the concrete every portion of the metal should be carefully examined, and wherever the paint or asphaltum has been scraped off in handling the iron should be thoroughly dried and the coating renewed.

Every pains should be taken to protect the beams from rusting for, when unprotected, steel beams rust very quickly, and if once the beams were subjected to the rusting process it would probably not be long before the building commenced to settle.*

Calculations for the Size of the Steel Beams.

A. Beams under a wall.—In determining the size of steel beams to be placed under a wall, as in Fig. 8, the first step is to determine the necessary width of the footing, which determines the length of the beams, and then the projection P may be fixed. The size of the beams depends upon the projection P and the load to be supported.

The width of the footing is obtained by dividing the load per lineal foot on the footing, by the safe resistance of the soil per square foot. This also gives the length of the beams.

* Several engineers advocate placing the beams without paint, believing that concrete is itself a better preservative than paint. The New York Building Code requires that the beams be painted, while the Chicago law does not

Knowing the length of the beams, the width of the masonry footing may be decided upon. The wider the stone footing, or the smaller the projection P, the less will be the transverse strain on the beams.

The beams are computed only for the portion projecting beyond the stone footing, as the load on the beams directly under the wall produces no transverse strain.

The beams are computed as if they resisted an upward pressure of the ground, as if they were supported as in Fig. 10 and

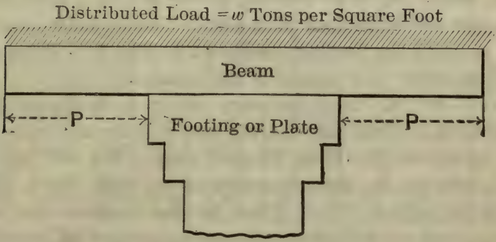

Fig. 10.

loaded with a distributed load equal, per square foot, to the safe resistance of the soil.

The formula for a beam loaded and supported in this way is that for a beam fixed at one end and uniformly loaded over its projection. The readiest method of computing the size of steel beams thus loaded and supported is to determine the necessary *coefficient of strength* for each beam, and then from the tables giving the strength of steel beams, find the size of beam having a coefficient equal to or next above the value determined. The *coefficient of strength* is given in the tables in Chapter XV, in the column headed C.

The necessary coefficient for the beams is found by the formula

$$C = 4 \times w \times P^2 \times s. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which w represents the assumed bearing power of the soil in tons per square foot, P the projection of the beam in feet, and s the spacing or distance, centre to centre, of beams, also in feet. $\times$ denotes multiplication.

Owing to the tendency of the beams, in bending, to concentrate the load on the outer edges of the masonry footing, and thus

crush them, which action would have the same effect on the beam as lengthening the arm or projection, the author recommends that when the course above the beams is of stone, brick, or concrete, at least one-third the width of the masonry footing *be added to the actual projection.*

In determining the width of the footing, or the area of a pier footing, the loads from the building should be computed as described on page 139. The calculations above indicated will be more clearly shown by the following example:

Example of Wall Footings.—A building is to be erected on a soil of which the safe bearing power is but 2 tons, and the pressure on each lineal foot of the stone footing is 20 tons. It is decided to build the footings as shown in Fig. 8. What should be the dimensions and weight of the beams?

Answer.—As the total pressure under each lineal foot of wall is 20 tons, and the safe bearing power of the soil is 2 tons, the footings must be $20 \div 2$, or 10 feet wide. We will use 4-foot granite blocks for the bottom course of the wall, which will give an actual projection (P) of 3 feet for the beams. For making the calculations we will add to the actual projection one-third of 4 feet or 16 inches, making the value of P $4\frac{1}{3}$ feet. We will assume 1 foot for the spacing of the beams, so that s will equal 1. The beams must then have a coefficient of strength $= 4 \times w \times P^2 \times s = 4 \times 2 \times (4\frac{1}{3})^2 \times 1 = 150.22$ tons. Examining the table giving the strength of standard steel I-beams (Chapter XV) we find that a 10-inch 35-pound steel beam has a coefficient of 156.2 tons, and a 25-pound beam 130.2 tons; therefore we must use 35-pound steel beams 10 feet long. If we spaced the beams 10 inches on centres, s would equal $\frac{5}{6}$ and C would equal $4 \times 2 \times (4\frac{1}{3})^2 \times \frac{5}{6}$, or 125.1 tons, which would enable us to use 25-pound beams, thereby effecting a saving of 50 pounds to the lineal foot of wall.

To save making the above calculations in each case the Carnegie Steel Company, Limited, publishes the following table from which the size of the beams may be taken direct.

To apply the table, look down the column having a heading equal to the resistance of the soil, and take the beam opposite the number equal to, or just above, the projection of the beam.

Thus in the above example $w = 2$ and the working projection is 4.33. The nearest projection above 4.33 (in the column headed 2, Table IV) is 4.90, which is opposite a 12-inch 31.5-lb. beam, which would be cheaper than the 10-inch 35-lb. beam.

To use the table for other values of s than 1 foot, w should be

TABLE IV, GIVING SAFE LENGTHS OF PROJECTIONS "P" IN FEET (SEE FIG. 8), FOR "s"=1 FOOT, AND VALUES OF "w," RANGING FROM 1 TO 5 TONS.

Depth of beam.	Weight per foot.	w (Tons per square foot).										
		1	1¼	1½	2	2¼	2½	3	3½	4	4½	5
In. 24	80.00	15.231	13.61	12.43	10.77	10.16	9.63	8.79	8.14	7.62	7.18	6.81
20	80.00	13.983	12.50	11.41	9.89	9.32	8.84	8.07	7.47	6.99	6.59	6.25
20	65.00	12.488	11.16	10.20	8.82	8.33	7.90	7.21	6.68	6.24	5.89	5.58
18	55.00	10.857	9.71	8.86	7.68	7.23	6.87	6.27	5.80	5.43	5.12	4.86
15	80.00	11.892	10.63	9.71	8.41	7.93	7.52	6.86	6.36	5.95	5.61	5.32
15	60.00	10.405	9.30	8.49	7.36	6.94	6.58	6.01	5.56	5.20	4.90	4.65
15	42.00	8.861	7.92	7.23	6.27	5.91	5.60	5.12	4.74	4.43	4.18	3.96
12	40.00	7.730	6.91	6.31	5.47	5.15	4.89	4.46	4.13	3.87	3.64	3.46
12	31.50	6.925	6.19	5.65	4.90	4.55	4.38	4.00	3.70	3.46	3.26	3.10
10	25.00	5.706	5.10	4.66	4.03	3.80	3.61	3.29	3.05	2.85	2.69	2.55
9	31.00	5.016	4.48	4.09	3.55	3.34	3.17	2.90	2.68	2.51	2.36	2.24
8	18.00	4.354	3.89	3.55	3.08	2.90	2.75	2.51	2.33	2.18	2.05	1.95
7	15.00	3.715	3.32	3.03	2.63	2.48	2.35	2.14	1.98	1.86	1.75	1.66
6	12.25	3.112	2.78	2.54	2.20	2.07	1.97	1.80	1.66	1.56	1.47	1.39
5	9.75	2.539	2.27	2.07	1.80	1.69	1.61	1.47	1.36	1.27	1.20	1.14
4	7.50	1.994	1.78	1.63	1.41	1.33	1.26	1.15	1.07	1.00	0.94	0.89

increased or decreased in the same ratio as s. Thus if $s=1\frac{1}{2}$ ft. w should be multiplied by $1\frac{1}{2}$. Taking $s=10''$, $w=2\times\frac{5}{6}=1\frac{2}{3}$. The projection under $1\frac{1}{2}$ opposite a 10-inch 25-lb. beam is 4.66, and as our projection was 4.33 it is evident that this beam would answer for a spacing of 10 ins.

In general, however, it will be better to calculate the beams by formula (1).

Beams under Piers.—In this case the size of the lower beams is determined in the same way as if under a wall, the length of P being taken from the end of the beam to the centre of the outer beam in upper tier.

For the upper beams the load borne by each beam should be computed and the coefficient of strength determined by the formula

$$C=4\times w\times P \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

w being in this case the total distributed load on either end of

the beam in pounds, and P the distance from end of beam to the base-plate above in feet.

Example.—The basement columns of a ten-story building are required to sustain a permanent load of 200 tons.

What should be the size of the beams in the footings, the supporting power of the soil being but 2 tons?

Answer.—Dividing the load by the bearing power of the soil we have 100 square feet, or 10×10 feet, for the area of the footing. We will arrange the beams as shown in Fig. 9, using a cast-iron bearing-plate 3 feet square under the column (instead of the stone block shown). The distance between the centres of outer beams in upper tier we will make 32 ins., thus making the value of P for the lower beams $= \dfrac{10'-2', 8''}{2}$, or $3\frac{2}{3}$ ft. s we will make 12 ins. or 1.

Then by formula 1, $C = 4 \times 2 \times (3\frac{2}{3})^2 \times 1 = 107.54$ tons, which is a little less than the coefficient for a 9-inch 25-lb. beam. As the 10-inch 25-lb. beam will cost no more and will be stiffer we should use that.

For the upper tier we will use five beams, spacing them 8 in. on centres. From an inspection of the plan it is evident that the five beams must support, or press down, an area equal to *abcd*, which in this case equals $3\frac{1}{2} \times 10$ ft., or 35 sq. ft. As the upward reaction is 2 tons per square foot, the five beams must be figured to support 70 tons (2×35), or 14 tons each. The projection will be $3\frac{1}{2}$ ft. Then by formula 2;—$C = 4 \times 14 \times 3\frac{1}{2} = 196$ tons. The coefficient for a 12-inch $31\frac{1}{2}$-lb. beam is 191.8, and for a 12-inch 40-lb. beam 239. As it is well to use heavy beams for the outer ones we will use two 12-inch 40-lb. beams and three 12-inch, $31\frac{1}{2}$-lb. beams in the upper tier.

If there were still another tier of beams the upper one would be calculated in the same way.

If the cap is of stone, the value of P should be taken at least 6 ins. greater than the actual projection, to allow of any crushing of the stone or mortar.

The deepest beam for the weight should always be used, and unless the beams in the upper tier have considerable excess of strength the two outer beams should be heavy beams.

When the footings carry iron or steel columns in the basement, as is generally the case, a cast-iron or steel base-plate should be used, as shown in Fig. 11. This plate should be bedded in Portland cement directly above the beams, as previously described.

In placing the beams it is essential that they be arranged symmetrically under the base-plate, otherwise they will sink more at one side than at the other.

Combined Footings.—Two columns are often supported on one footing, as shown in Fig. 11, and quite often four columns, one near each corner. The computations for combined footings are more complicated than for simple footings, especially when the columns are unequally loaded, and require a considerable knowledge of mechanics. The best presentation of the subject with which the author is acquainted is in Freitag's *Architectural Engineering*, second edition.

Description of Steel-beam Footings.—A very good idea of what has been done in the way of supporting buildings on spread footings of steel beams and girders, and of the various arrangements that have been employed, may be obtained from the descriptions of actual construction referred to below:

Surface Foundations, *Engineering Record*, July 2, 1898.

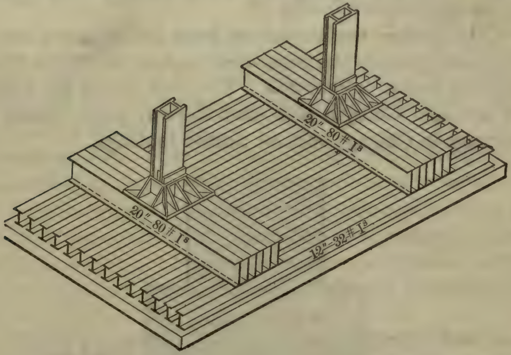

Fig. 11.

Foundations of High Buildings, by W. B. Hutton, *Engineering Record*, September 23, 1893.

St. Paul Building, New York City, *Engineering Record*, June 25, 1898.

Harrison Building, Philadelphia, *Engineering Record*, Aug. 22, 1896.

Franklin Building (9 to 15 Murray Street), New York, *Engineering Record*, May 28, 1898.

Buek Building, New York City, 25 ft. front, *Engineering Record*, June 25, 1898.

Masonic Temple, Chicago, *Engineering Record*, Aug. 13, 1898.

De Dino Building, New York City, *Engineering Record*, Aug. 13, 1898.

The Wilkes Building, New York City, *Engineering Record*, June 7, 1890.

American Exchange Bank, New York City, *Engineering Record*, Oct. 14, 1899.

Timber Footings.—For buildings of moderate height timber may be used for giving the necessary spread to the footings, *provided water is always present.* The footings should be built by covering the bottom of the trenches, which should be perfectly level, with 2-inch plank laid close together and longitudinally of the wall. Across these heavy timbers should be laid, spaced about 12 inches from centres, the size of the timbers being proportioned to the transverse strain. On top of these timbers again should be spiked a floor of 3-inch plank of the same width as the masonry footings which are laid upon it. A section of such a footing is shown in Fig. 12.

All of the timber work must be kept below low-water mark, and the space between the transverse timbers should be filled with sand, broken stone, or concrete. The best woods for such foundations are oak, Georgia pine, and Norway pine. Many of the old buildings in Chicago rest on timber footings.

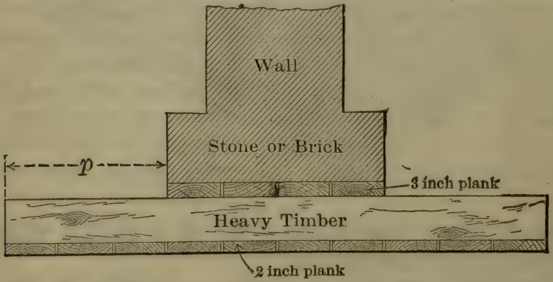

Fig. 12

Calculation for the Size of the Cross Timbers.—The size of the transverse timbers should be computed by the following formula:

$$\text{Breadth in inches} = \frac{2 \times w \times P^2 \times s}{D^2 \times A}, \qquad (3)$$

w representing the bearing power in pounds per square foot, P

the projection of the beam beyond the 3-inch plank in feet, s the distance between centres of beams *in feet*, and D the assumed depth of the beam in inches. A is the constant for strength, and should be taken at 90 for Georgia pine, 65 for oak, 60 for Norway pine, and 55 for common white pine or spruce.

Example.—The side walls of a given building impose on the foundation a pressure of 20,000 pounds per lineal foot; the soil will only support, without excessive settlement, 2,000 pounds to the square foot. It is decided for economy to build the footings as shown in Fig. 12, using Georgia pine timber. What should be the size of the transverse timbers?

Answer.—Dividing the total pressure per lineal foot by 2,000 pounds, we have 10 feet for the width of the footings. The masonry footing we will make of granite or other hard stone, 4 feet wide, and solidly bedded on the plank in Portland-cement mortar. The projection P of the transverse beams would then be 3 feet. We will space the beams 12 inches from centres, so that $s=1$, and will assume 10 inches for the depth of the beams. Then by formula (3), breadth in inches $=\dfrac{2\times 2000\times 9\times 1}{100\times 90}=4$, or we should use $4''\times 10''$ timbers, 12 inches from centres. If common pine timber were used we should substitute 55 for 90, and the result would be $6\frac{1}{2}$.

Foundations for Temporary Buildings.—When temporary buildings are to be built over a compressible soil, the foundations may, as a rule, be constructed more cheaply of timber than of any other material, and in such cases the durability of the timber need not be considered, as good sound lumber will last two or three years in almost any place if thorough ventilation is provided.

The World's Fair buildings at Chicago (1893) were, as a rule, supported on timber platforms, proportioned so that the maximum load on the soil would not exceed $1\frac{1}{4}$ tons per square foot. Only in a few places over "mud-holes" were pile foundations used.*

Masonry Wells for Foundations.—Where the site of the building is composed of compressible soil overlaying bed-rock or hard-pan, and especially where the site has been filled or the conditions are not suitable for piling, wells of masonry

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* A description of the foundations of these buildings may be found in Part I, "Building Construction and Superintendence," p. 55.

sunk to the bed-rock or hard-pan will generally prove as cheap as any other equally good foundation.

The wells are formed by driving cylindrical tubes of from 4 to 6 ft. in diameter through the soil to the bearing stratum. The tubes are usually made in short lengths and spliced as they are sunk. After the tube has reached the firm stratum, it is excavated and filled with brickwork or concrete, the masonry being intended to support the weight, while the steel shell merely forms a wall around the pier and enables it to be built.

The wells are arranged as isolated piers, with the walls of the superstructure supported by steel girders resting on the piers.

A notable example of this type of foundation is that of the City Hall of Kansas City, Mo.*

Such wells were also used under the new Stock Exchange in Chicago. For the new Stock Exchange Building in New York wooden cylinders were employed.

Caisson Foundations.—In the case of the tall buildings of New York City, which as a rule are built over a soil composed of mud and quicksand, it has been found, in many cases, impossible to safely support the building on the natural soil and caissons sunk to the bed-rock by the pneumatic process have been resorted to as the most satisfactory method of obtaining a foundation.

Caissons have been used for many years in building the foundations of bridges, but the first instance of their use for buildings is believed to be in the foundations of the Manhattan Life Insurance Company's building, New York City, in 1893. Since that time caissons have been used in providing the foundation for several buildings in that city. Caissons as used in building foundations are made both cylindrical and rectangular in shape, and they have been built both of wood and steel, the latter material being more commonly used. Cylindrical caissons are the most convenient and economical, but the positions of the columns and the necessity of supporting two and often four columns on the same caisson usually make it necessary to use rectangular caissons.

The size of the caissons vary according to the load and number of columns to be supported. Caissons as small as 5 ft. in diameter have been used, although from 8 to 10 ft. is a more common

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* For illustrations, see Part I, "Building Construction and Superintendence."

size for cylindrical caissons. Rectangular caissons have been used as large as $15\frac{1}{2} \times 25$ ft. in plan. The usual height of the caissons that have thus far been used is from 11 to 12 ft.

The caissons are sunk until they reach bed-rock (which lies from 50 to 60 feet below the Broadway street-level in New York); the surface of the rock is then cleaned and dressed to level surfaces and the caissons rammed full of concrete. On top of the caissons, piers of hard-burned bricks are built to the proper height to receive the superstructure, the piers being generally built as the caissons descend, so that the top of the masonry will always be above the water-line. The weight of the pier assists in sinking the caisson.

"Although this process is costly, it has proved reliable and applicable under the most troublesome conditions for carrying masonry piers to solid rock at depths as great as 100 feet below the water-line, although such great distances have not yet been required for buildings. One of the greatest advantages claimed for this method is the care and precision which can be exercised in preventing the inflow of quicksand and outside materials and thus avoiding any disturbance of the equilibrium in the surrounding soils or settlements of adjacent loaded piers or the undermining of walls. The pneumatic caisson consists of a steel or wooden box with vertical sides and a flat top, but no bottom. Its lower edges are provided with a cutting edge and it is made air-tight and filled with air under any required pressure, which is maintained by a powerful steam pump. Access is had to the interior or working chamber through an extensible vertical shaft in the roof surmounted by a small chamber or air lock with two doors, the outer of which is closed whenever the inner one is opened to give access to the shaft. As both doors are never opened simultaneously, no direct communication is established between the atmosphere and the interior air pressure, and only a small quantity of compressed air is lost at each opening of the outside door.

Two or more shafts and air locks are usually provided for materials and for the workmen. The doors of the material lock are successively opened and closed as quickly as possible, but in the man lock the operation is a gradual one, because the pressure in the lock must be slowly increased or diminished to avoid injurious effects to the inmates. The men in the working chamber excavate the earth underneath it and undermine its edges so that it gradually sinks under the increasing load of the brick or stone

masonry built up on the roof or heavy deck which forms the top of the working chamber. The excavated material is hoisted to the surface of the ground either in buckets through the material locks, or, when it is loose earth or mud, is blown up with water through vertical pipes open on top and having their lower ends sealed below the surface of the water in the interior of the caisson. The caissons are lighted by electricity and often have telephone communication with the superintendent's office above. Excavation is carried on by pick and shovel, and when necessary by blasting with dynamite. Except at considerable depths the men work the usual number of hours without experiencing much evil effects from the increased pressure." *

A complete description of the foundation of the Manhattan Life Insurance Building (N. Y.) may be found in the *Engineering Record* of Jan. 20, 1894, and an abstract of the same in "Building Construction and Superintendence," Part I.

Descriptions of other caisson foundations may be found in the following numbers of the *Engineering Record*, also in Freitag's "Architectural Engineering."

July 13, 1896, American Surety Building, New York.

July 11, 1896, The Standard Block, (N. Y.).

Jan. 16, 1897, The Gillender Building (N. Y.), timber caisson.

Nov. 27, 1897, Five-foot cylindrical steel caissons.

Dec. 11, 1897, Empire Building (N. Y.)

Dec. 10, 1898, Residence (N. Y.) cylindrical wood caissons.

Oct. 28, 1898, McCready Building (N. Y.), cylindrical wood caissons.

Sept. 28, 1901, Stock Exchange, N. Y., wood caissons, rectangular and cylindrical.

Cantilever Foundations.

When buildings of skeleton construction are erected without a party wall agreement, it is usually impossible to obtain a symmetrical foundation directly under the columns supporting the side or party wall, and in such cases the foundation piers are commonly built sufficiently inside of the wall line to give the necessary spread to the footings, and at the same time have them symmetrical with regard to the centre of pressure. Cantilever girders resting on these piers as a fulcrum are then used to

* *Engineering Record*, July 30, 1898.

carry the column next to the building line. By this method, also, it is sometimes possible to build without undermining the adjoining property.

Various arrangements of cantilevers have been used during the past ten years, the particular arrangement being usually determined by some peculiarity of the column groupings, or relation to adjoining building.

Figs. 13, 14, and 15 * show three different designs which illustrate fairly well the different types of cantilevers as used in foundations.

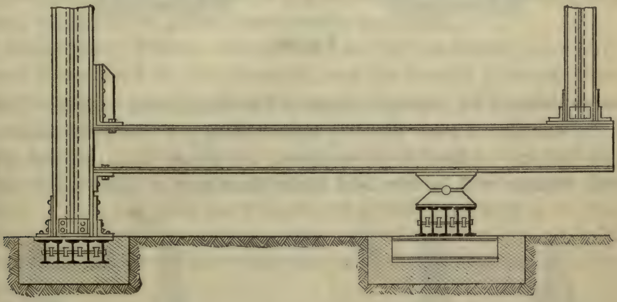

Fig. 13.

Fig. 13 shows deep steel beams, used when the load on the column resting on the cantilever produces such bending moments as can be taken up by the beams. In this type the long end of the cantilever is connected to an interior column by means of riveted connections.

Fig. 14 shows a method of cantilever construction where it is not desirable to have a separate foundation under each column, and a heavy box girder of suitable design is used to transmit the various column loads to two independent foundations. Owing to danger of unequal settlement in the supporting piers, which would affect the stresses in the girder, this form of girder should be avoided if possible.

Fig. 15 illustrates one of the latest types of cantilever foundations, in which the objection to the continuous girder is overcome.

* From the "Pocket Companion" of the Carnegie Steel Co., Limited, by permission.

"An important feature in connection with cantilever construction is to adopt a pin support in place of resting the cantilever

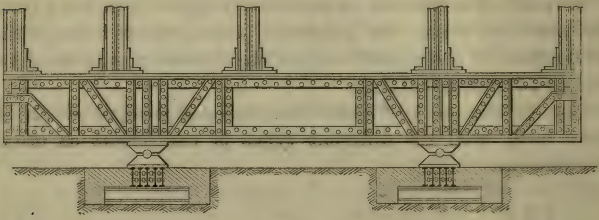

Fig. 14.

beam directly on the top course of the foundation beams. For, if the cantilever rests directly upon the upper course of foundation beams, without a pin support, the outer beam nearest the wall column will be strained more than any of the others, and

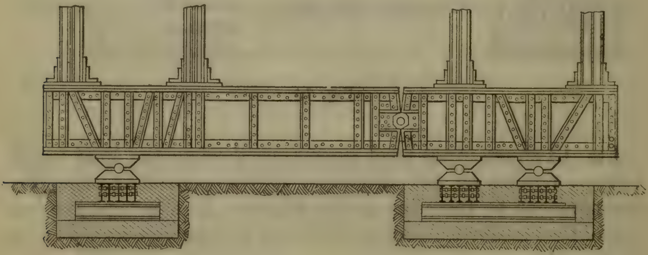

Fig. 15.

thus the centre of pressure will not be exactly in the middle of the foundation, as it should be.

"The shoes for ordinary loads and conditions are made solid of cast iron, and the pin of steel.

"The height of each shoe should not be less than 6 ins. and the pin $2\frac{1}{2}$ ins. in diameter. Each individual case should be figured by itself, the pin being figured for bearing, or crushing only.

"A clearance of $\frac{1}{2}''$ to $1''$ is allowed between the cast shoes, which are always faced, and the hole bored to suit the pin." *

* F. H. Kindl, C.E.

Illustrated descriptions of cantilever foundations may be found in the following numbers of the *Engineering Record*:

Cantilever Foundations for Small Buildings, Nov. 27, 1897.

Exchange Court Building, New York City, June 11, 1898.

Developments of Architectural Construction, July 30, 1898.

The calculations for cantilever foundations involve the determination of bending moment, shearing, and bucking in the girder and the reaction on the fulcrum and at the long end of the girder.

The arrangements are so numerous that no special rules can be given, but each case must be calculated by means of the general principles relating to the strength of girders, and for determining supporting forces, given elsewhere in this book.

Concrete Piles.—Since the year 1902 concrete piles have been introduced in this country, and for several years previous they had been used to some extent in Europe. The practice of French and German engineers has been to construct the piles on the ground, casting them in a mould in which a steel skeleton for reinforcement is first inserted and also a steel or cast-iron shoe. After the piles have hardened a sufficient length of time they are driven like timber piles, a cushioned cap being placed on top of the pile to distribute the force of the blow evenly over the concrete.*

In this country the practice thus far has been to construct the piles in place by driving a hollow steel cylinder which retains the walls of the hole in place until the concrete has been deposited and rammed. As the filling-in of the concrete progresses, the shell is drawn up, the lower end of the shell being always about 6 ins. below the top of the concrete. If desired, a reinforcing skeleton can be placed in the shell before the concrete is poured, but the piles are very strong without it.

Thus far two styles of shells have been successfully used, viz., the "Simplex," controlled by the Simplex Concrete Piling Co. of Philadelphia, and the Raymond Pile, controlled by the Raymond Concrete Pile Co. of Chicago, from whom complete information as to cost, carrying capacity, etc., may be obtained.

Concrete piles, although more expensive than timber piles, possess many advantages over the latter, and can be used in places where timber piles would not be durable.

They are capped with concrete or steel grillage in the same manner as described for timber piles.

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* For description of several types of cast piles, see the *Engineering News* of March 10, 1904.

CHAPTER III.

MASONRY WALLS AND FOOTINGS.

CEMENTS AND CONCRETES.

Footing Courses.—Every foundation or bearing wall overlaying anything except solid rock should rest on a footing or base projecting beyond the wall on each side. On wet or compressible soils these footings may be built of steel beams and concrete, concrete and twisted iron, or timbers, as described in Chapter II, but on firm soils the footings are almost invariably either of concrete, stone, or brick.

Footings answer two important purposes:

1st. By distributing the weight of the structure over a larger area of bearing surface, the pressure per square foot on the ground is diminished and the liability to vertical settlement correspondingly lessened.

2d. By increasing the area of the base of the wall they add to its stability and form a protection against the danger of the work being thrown out of plumb by any forces that may act against it. Nearly every building law requires that every foundation wall and every pier shall have a footing at least 12 inches wider (6 ins. on each side) than the thickness of the wall or pier, and this may be considered as the minimum projection, except in rare instances where there may be a special reason for making it less. On firm soils a projection of 6 ins. on each side of the wall will generally reduce the unit pressure * to a point within the safe resistance of the soil, but it is always wise to proportion the footings to a uniform unit pressure, as explained on pages 137–139.

To have any useful effect, footings must be well bedded and have sufficient transverse strength to resist the upward reaction on the projection.

Stone Footings.—Stone foundation walls generally have stone footings, although if the wall is heavily loaded a bottom

* Pressure per square foot.

footing of concrete is advisable under the stone footing. If practicable, stone footings should consist of stones having a width equal to that of the footing. If impracticable to obtain stones of this size, then two stones should be used, meeting under the centre of the wall. In any event the footing courses should extend inside of the course above, a distance equal to at least $1\frac{1}{2}$ times the projection, otherwise the stones will not be able to transmit the necessary pressure, but will open at the joints as in Fig. 1.

Stone footings should be of hard, strong, and durable stone, always laid on their natural bed and be solidly bedded in mortar. As a general rule, the thickness of each course should be about equal to its projection beyond the course above. The most common defect in large stone footings is that the stones are not properly bedded, it being more difficult to bed a large stone than a small one. The stones should be laid in a thick bed of mortar and worked with a bar sideways until firmly settled into the mortar.

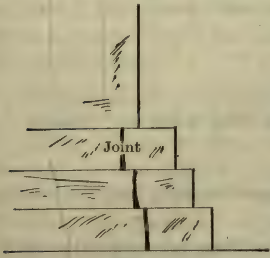

Fig. 1.

Offsets.—The projection of the footings beyond the wall, or the course above, is a point that must be carefully considered, whatever be the material of the footings.

If the projection of the footing or offset of the courses is too great for the strength of the stone, brick, or concrete, the footing will crack, as shown in Fig. 2.

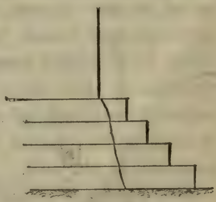
Fig. 2.

The proper offset for each course will depend upon the vertical pressure, the transverse strength of the material, and the thickness of the course. Each footing stone may be considered as a beam fixed at one end and uniformly loaded, and in this way the safe projection may be calculated.

Table I gives the *safe offset for masonry footing courses, in terms of the thickness of the course*, computed by a factor of safety of 10.

It should be borne in mind that as each footing course transmits the entire weight of the wall and its load, the pressure will

TABLE I.

Kind of footing.	R. in lbs. per sq. in.*	Offset for a pressure, in tons per sq. ft. on the bottom of the course, of 0.5	1	2	3	5	10
Bluestone flagging.	2700	3.6	2.6	1.8	1.5	1.2	.8
Granite.	1800	2.9	2.1	1.5	1.2	1	.7
Limestone.	1500	2.7	1.9	1.3	1.1	.9	.6
Sandstone.	1200	2.6	1.8	1.3	1.0	.8	.5
Slate.	5400	5.0	3.6	2.5	2.2	1.5	1.2
Best hard brick.	1200	2.6	1.8	1.3	1.0	.8	.5
Concrete { 1 Portland. 2 sand. 3 pebbles.	150	0.8	0.6	0.4			
Concrete { 1 Rosendale. 2 sand. 3 pebbles.	80	0.6	0.4	0.3			

* Modulus of Rupture, values given by Prof. Baker in "Treatise on Masonry Construction."

be greater per square foot on the upper courses, and the offsets should be made proportionately less.

Concrete Footings.—For all buildings of any great weight, and especially if built on a clay soil, the author believes that cement concrete makes the best footing, and that it is even preferable to and generally cheaper than large slabs of stone. When the concrete is properly made and used, it attains a strength equal to that of most stones, and being devoid of joints, it is like a continuous beam, having sufficient strength to span any soft spots that might happen to be in the foundation. When deposited in thin layers and well rammed the concrete also becomes firmly bedded on the bottom of the trenches, so that there is no possible chance for settlement except that due to the compression of the soil.

For footings, concrete should always be mixed with cement, preferably Portland cement, and should have a thickness of at least 8 ins., even under light buildings, and for buildings of more than two stories, a thickness of at least 12 ins. In firm soils, such as clay, the trenches should be accurately dug and trimmed to the exact width of the footing, so that the concrete will fill the trench. When the soil is of loose gravel or sand it is generally necessary to set up planks to confine the concrete and form the sides of the footings. These planks may be held in place by stakes; they should be left in place until the concrete has become hard, which generally requires from two to four days,

after which they may be pulled up and dirt filled in against the concrete.

The proportions and manner of mixing concrete are described in the latter part of this chapter.

Concrete should be used as soon as mixed and should always be deposited in layers, which as a rule should not exceed 6 ins. in thickness, especially for the first layer. On small jobs where the work is done by hand the concrete is usually carried to the trenches in wheel-barrows and dumped into the trench. The height from which the concrete is dumped, however, should not exceed 4 feet above the bottom of the trench, as when falling from a greater height the heavy particles are apt to separate from the lighter ones.

As soon as the concrete has been deposited in the trench, it should be levelled off and then tamped with a wooden rammer weighing about 20 lbs., until the water in the concrete is brought to the surface. Concrete should not be permitted to dry too quickly, and if twenty-four hours elapse between depositing the successive layers, the top of each layer should be sprinkled before the next is deposited.

For buildings over five stories high, it is a good idea to place a stone footing above the concrete footing, if suitable stones for the purpose can be obtained.

Brick Footings.—Where the foundation walls are of brick, the footings are usually either of brick or concrete. For interior

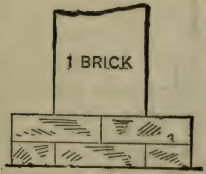

Fig. 3

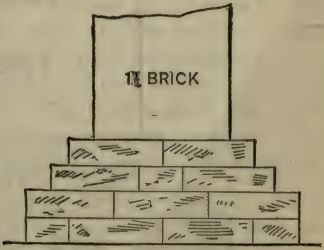

Fig. 4

walls on dry ground, and in many localities for outside walls, brick footings are fully as good as stone footings, provided good hard bricks are used and the footings are properly built.

Brick footings should always start with a double course and then be laid in single course for ordinary footings, the outside

of the work being laid all headers, as in the accompanying illustrations, and no course projecting more than one fourth brick beyond the one above it, except in the case of an 8- or 9-inch wall. For brick footings under high or heavily loaded walls, each projecting course should be made double, the heading course above and the stretching course below. Figs. 3, 4, 5, and 6 show footings for walls varying from one brick to three bricks in thickness.

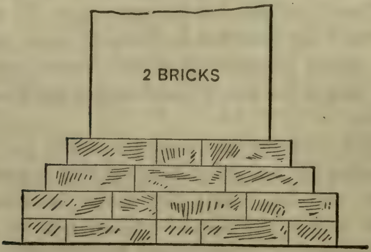

Fig. 5

The bricks used for footings should be the hardest and soundest that can be obtained, and should be laid in cement or hydraulic lime mortar, either grouted or thoroughly slushed up, so that every joint shall be entirely filled with mortar. The writer favors grouting brick footings, that is, using thin mortar for

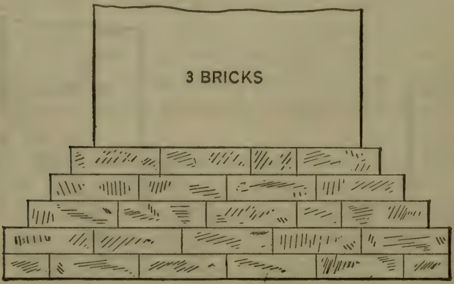

Fig. 6

filling the inside joints, as he has always found it to give very satisfactory results.

The bottom course of the footing should always be laid in a bed of mortar spread on the bottom of the trench, after the latter

has been carefully levelled. All bricks laid in warm or dry weather should be thoroughly wet before laying, for, if laid dry, the bricks will rob the mortar of a large percentage of the moisture it contains, greatly weakening the adhesion and strength of the mortar.

Too much care cannot be bestowed upon the footing courses of any building, as upon them depends much of the stability of the work. If the bottom courses be not solidly bedded, if any rents or vacuities are left in the beds of the masonry, or if the materials themselves be unsound, or badly put together, the effects of such carelessness are sure to show themselves sooner or later, and almost always at a period when remedial efforts are difficult and expensive.

Inverted Arches.—When the external walls of a building are divided into piers, with wide openings between, and the supporting power of the soil is not more than two or three tons to the square foot it may be desirable to connect the base of the piers by means of inverted arches, for the purpose of distributing the weight of the piers over the whole length of the footings.

Examples of inverted arch footings are shown by figures 7 and 8,* which represent respectively the construction employed in the Drexel Building in Philadelphia and the World Building in New York.

Unless the piers are about equally loaded, however, it will be difficult to distribute the weight evenly, and if the arches extend to an angle of the building, the end arch must be provided with ties of sufficient strength to resist the thrust of the arch, otherwise it may push out the corner pier. In the opinion of the author, it is usually better to build the piers with separate footings, projecting equally on all four sides of the pier, and each proportioned to the load supported. The intermediate wall may be supported either by steel beams or arches as preferred. An example showing the method of proportioning inverted arches is given in Chapter III. of Part I., "Building Construction."

Foundation Walls.

This term is generally applied to those walls which are below the surface of the ground, and which support the superstructure. Walls whose chief office is to withhold a bank of earth, such as around areas, are called retaining walls.

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* From the *Engineering Record* of May, 1890, and Nov., 1890.

Foundation walls may be built of brick, stone or concrete, the former being the most common. Brick walls for foundations are only suitable in very dry soils or in the case of party walls, where there is a cellar or basement on each side of them.

Portland cement concrete is an excellent material for foundation walls, and is being more extensively used for that purpose

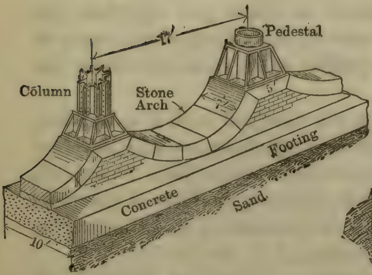

Fig. 7

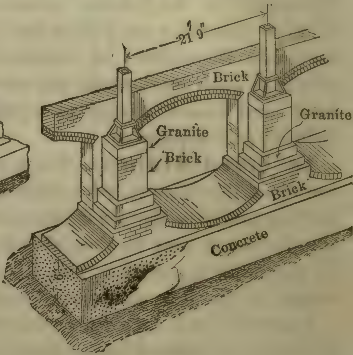

Fig. 8

every year. The concrete may be filled in between wooden forms, which hold it in place until the cement has set, or concrete blocks moulded so as to form a solid wall may be used.

If poured concrete is used the forms should be removed as soon as the cement has set and the walls sprinkled once or twice a day, if the weather is dry, so that the concrete will not dry too quickly.

Good hard ledge stone, especially if it comes from the quarry with flat beds, not only makes a strong wall but if well built, one that will stand the effects of moisture and the pressure of the earth much better than a brick wall. As between a good stone wall and a wall of Portland cement concrete, there is probably not much choice, except perhaps in the matter of expense, the relative cost of stone work and concrete varying in different localities. A wall built of soft stone, or stone that is very irregular in shape, with no flat surfaces, is greatly inferior to a concrete wall, or even to a wall of good hard brick, and should be used only for dwellings or light buildings. Stone walls should never be less than 18 ins. thick, and should be well bonded, with full and three quarter headers, and all spaces between the stones filled solid with mortar and broken stone or spauls.

The mortar for stone work should be made of hydraulic lime or cement, and sharp and rather coarse sand.

The outside walls of cellars and basements should be plastered smooth on the outside with 1 to 2, or 1 to $1\frac{1}{2}$ cement mortar, $\frac{1}{2}$ inch to $\frac{3}{4}$ inches thick.

In heavy clay soils it is a good idea to batter the walls on the outside, making the wall from 6 ins. to a foot thicker at the bottom than at the top.

The thickness of the foundation wall is usually governed by that of the walls above, and also by the depth of the wall.

Nearly all building regulations require that the thickness of the foundation wall, to the depth of 12 ft. below the grade line, shall be 4 ins. greater than the wall above for brick and 8 ins. for stone, and for every additional 10 ft., or part thereof deeper, the thickness shall be increased 4 ins. In all large cities the thickness of the walls is controlled by law. For buildings where the thickness is not so governed the following table will serve as a fair guide:

TABLE II.—THICKNESS FOR FOUNDATION WALLS.

Height of building.	Dwellings, Hotels, etc.		Warehouses.	
	Brick.	Stone.	Brick.	Stone.
	Inches.	Inches.	Inches.	Inches.
Two stories...........	12 or 16	20	16	20
Three stories..........	16	20	20	24
Four stories...........	20	24	24	28
Five stories...........	24	28	24	28
Six stories............	28	32	28	32

Brick and Stone Walls.—Very little is known regarding the stability of walls of buildings beyond what has been gained by practical experience. The only strain which comes upon any horizontal section of such a wall, which can be ascertained, is the direct weight of the wall above, and the pressure due to the floors and roof.

In most walls, however, there is more or less tendency to buckle, to overcome which it is necessary to make the walls thicker than would be required to resist the direct crushing stress. The resistance to fire should also be taken into account in deciding on the thickness of any given wall.

The strength of a wall also depends very much upon the quality of the materials used and upon the way in which the wall is built.

A wall bonded every twelve inches in height, and with every joint slushed full with good rich mortar, will be as strong as a poorly built wall four inches thicker. Walls laid with cement mortar are also much stronger than those laid with lime mortar, and a brick wall built with bricks that have been *well wet* just before laying is *very much stronger* than one built with dry bricks.

Thickness of External Walls.—In nearly all the larger cities of the country the minimum thickness of the walls is prescribed by law or ordinance, and as these requirements are generally ample they are commonly adhered to by architects when designing brick buildings. Table III. gives a comparison of the thickness of brick walls required for mercantile buildings in the representative cities of the different sections of the United States, and affords about as good a guide as one can have because the values given, as a rule, represent the judgment of well qualified and experienced persons.

Walls for dwellings are generally permitted to be 4 ins. less in thickness than for warehouses, although in some cities little or no distinction is made between business blocks and dwellings.

Table IV. gives the thickness required for the brick walls of dwellings, tenements, hotels, and office buildings in the city of Chicago, which is as light as such walls should be built. Most cities require 13-inch walls in the upper story of three-story buildings, and for large two-story dwellings.

In St. Louis the top two stories of dwellings are required to be 13 ins. thick, the next two, below, 18 ins. thick, the next two 22 ins., and the next two 26 ins.

In compiling Table III. the top of the second floor was taken at 19 ft. above the sidewalk, and the height of the other stories at 13 ft. 4 ins., including the thickness of the floor, as the New York and Boston laws give the height of the walls in feet instead of in stories. When the height of stories exceeds these measurements the thickness of the walls in some cases will have to be increased.

The maximum height of stories permitted by the Chicago ordinance with these thicknesses of walls is 18 ft. in first story, 15 ft. in second story, 13 ft. 6 ins. in the third, and 12 ft. in the stories above.

TABLE III.—THICKNESS OF WALLS IN INCHES, FOR MERCANTILE BUILDINGS AND PUBLIC STABLES, AND, EXCEPT IN CHICAGO, FOR ALL BUILDINGS OVER FIVE STORIES IN HEIGHT.

Height of Building.		Stories.							
		1st.	2d.	3d.	4th.	5th.	6th.	7th.	8th.
Two Stories.	Boston	16	12						
	New York	12	12						
	Chicago	12	12						
	Minneapolis	12	12						
	St. Louis	18	13						
	Denver	13	13						
	San Francisco	17	13						
	New Orleans	13	13						
Three Stories.	Boston	20	16	16					
	New York	16	16	12					
	Chicago	16	12	12					
	Minneapolis	16	12	12					
	St. Louis	18	18	13					
	Denver	17	17	13					
	San Francisco	17	17	13					
	New Orleans	13	13	13					
Four Stories.	Boston	20	16	16	16				
	New York	16	16	16	12				
	Chicago	20	16	16	12				
	Minneapolis	16	16	12	12				
	St. Louis	22	18	18	13				
	Denver	21	17	17	13				
	San Francisco	17	17	17	13				
	New Orleans	18	18	13	13				
Five Stories.	Boston	20	20	20	20	16			
	New York	20	16	16	16	16			
	Chicago	20	20	16	16	16			
	Minneapolis	20	16	16	12	12			
	St. Louis	22	22	18	18	13			
	Denver	21	21	17	17	13			
	San Francisco	21	17	17	17	13			
	New Orleans	18	18	18	13	13			
Six Stories.	Boston	24	20	20	20	20	16		
	New York	24	20	20	20	16	16		
	Chicago	20	20	20	16	16	16		
	Minneapolis	20	20	16	16	16	12		
	St. Louis	26	22	22	18	18	13		
	Denver	26	21	21	17	17	13		
	San Francisco	21	21	17	17	17	13		
	New Orleans	22	18	18	18	13	13		
Seven Stories.	Boston	24	20	20	20	20	20	16	
	New York	28	24	24	20	20	16	16	
	Chicago	20	20	20	20	16	16	16	
	Minneapolis	20	20	20	16	16	16	12	
	St. Louis	26	26	22	22	18	18	13	
	Denver	26	21	21	21	17	17	17	
	New Orleans	22	22	18	18	18	13	13	
Eight Stories.	Boston	28	24	20	20	20	20	20	16
	New York	32	28	24	24	20	20	16	16
	Chicago	24	24	20	20	20	16	16	16
	Minneapolis	24	20	20	20	16	16	16	12
	St. Louis	30	26	26	22	22	18	18	13
	Denver	30	26	21	21	21	17	17	17
	New Orleans	22	22	22	18	18	18	13	13

Height of Building.		Stories.											
		1st.	2d.	3d.	4th.	5th.	6th.	7th.	8th.	9th.	10th.	11th.	12th.
Nine Stories.	Boston.	28	24	24	20	20	20	20	20	16			
	New York. . .	32	32	28	24	24	20	20	16	16			
	Chicago.	24	24	24	20	20	20	16	16	16			
	Minneapolis .	24	24	20	20	20	16	16	16	12			
	St. Louis. . . .	30	30	26	26	22	22	18	18	13			
	Denver.	30	26	26	21	21	21	17	17	17			
Ten Stories.	Boston.	28	28	24	24	20	20	20	20	20	16		
	New York. . .	36	32	32	28	24	24	20	20	16	16		
	Chicago.	28	28	24	24	24	20	20	20	16	16		
	Minneapolis.	24	24	24	20	20	20	16	16	16	12		
	St. Louis. . . .	34	30	30	26	26	22	22	18	18	13		
	Denver.	30	30	26	26	21	21	21	17	17	17		
Eleven Stories.	Boston.	36	32	32	28	28	24	20	20	20	20	16	
	New York. . .	36	36	32	28	28	24	24	20	20	16	16	
	Chicago.	28	28	24	24	24	20	20	20	16	16	16	
	St. Louis. . . .	34	34	30	30	26	26	22	22	18	18	13	
	Denver.	30	30	26	26	26	21	21	21	17	17	17	
Twelve Stories.	Boston.	36	36	32	32	28	28	24	20	20	20	20	16
	New York. . .	40	36	36	32	32	28	24	24	20	20	16	16
	Chicago.	28	28	28	24	24	24	20	20	20	16	16	16
	St. Louis. . . .	34	34	34	30	30	26	26	22	22	18	18	13
	Denver.	30	30	30	26	26	26	21	21	21	17	17	17

TABLE IV.—THICKNESS OF ENCLOSING WALLS, FOR RESIDENCES, TENEMENTS, HOTELS, AND OFFICE BUILDINGS.—CHICAGO BUILDING ORDINANCE.

	Basement.	Stories.											
		1st.	2d.	3d.	4th.	5th.	6th.	7th.	8th.	9th.	10th.	11th.	12th.
Basement and.	12	8											
Two-story.	12	12	8										
Three-story.	16	12	12	8									
Four-story.	20	16	16	12	12								
Five-story.	20	16	16	16	12	12							
Six-story.	20	20	16	16	16	12	12						
Seven-story.	24	24	20	20	16	16	12	12					
Eight-story.	24	24	24	20	20	16	16	12	12				
Nine-story.	28	24	24	20	20	20	16	16	12	12			
Ten-story.	28	24	24	24	20	20	20	16	16	12	12		
Eleven-story.	28	28	24	24	24	20	20	20	16	16	12	12	
Twelve-story.	32	28	28	24	24	24	20	20	20	16	16	12	12

General Rule for Thickness of Walls.—Although there is a great difference in the thicknesses given in Table III., more indeed than there should be, yet a general rule might be de-

duced from the table, for mercantile buildings over four stories in height, which would be somewhat as follows:

For brick equal to those used in Boston or Chicago, make the thickness of the three upper stories 16 ins., of the next three below 20 ins., the next three 24 ins., and the next three 28 ins. For a poorer quality of material make only the two upper stories 16 ins. thick, the next three 20 ins., and so on down.

In buildings less than five stories in height the top story may be 12 ins. in thickness.

In determining the thickness of walls the following general principles should be recognized:

First. That walls of warehouses and mercantile buildings should be heavier than those used for living or office purposes.

Second. That high stories and clear spans exceeding 25 ft. require thicker walls.

Third. That the length of the wall is a source of weakness, and that the thickness should be increased 4 ins. for every 25 ft. over 100 or 125 ft. in length. (In New York the thickness in the table must be increased for buildings exceeding 105 ft. in depth. In Western cities the tables are compiled for warehouses 125 ft. in depth, as that is the usual depth of lots in those cities.)

Fourth. That walls containing over 33 per cent. of openings should be increased in thickness.

Fifth. Partition walls may be 4 ins. less in thickness than the outside walls if not over 60 ft. long, but no partition to be less than 8 ins. thick.

Walls Faced with Ashlar.—"In reckoning the thickness of walls, no allowance shall be made for ashlar, unless it is 8 ins. or more thick, in which case the excess over 4 ins. shall be reckoned as part of the thickness of the wall. Ashlar shall be at least 4 ins. thick and properly held by metal clamps to the backing, or properly bonded to the same."—Boston Building law.

Stone Walls should generally be 4 ins. thicker than required for brick walls.

Hollow Walls.—Hollow walls are undoubtedly desirable for dwellings, and might well be used for other buildings not more than four or five stories in height, on account of the security afforded from the weather. Owing to the fact that they are usually more expensive than solid walls, and occupy more space, they are not very extensively used in this country, except in concrete construction.

The Boston building law requires that "vaulted walls shall contain, exclusive of withes, the same amount of material as is required for solid walls, and the walls on either side of the air-space shall be not less than 8 ins. thick, and shall be securely tied together with ties not more than 2 ft. apart."

For a description of the construction of hollow brick walls, see Part I of "Building Construction."

Walls of Cement Blocks.—Blocks made of Portland-cement concrete, and formed in moulds, are rapidly coming into use for building walls and partitions. Within the past two years several patents have been taken out on different forms of blocks and on machines or processes for making the same, and many buildings have been erected or are now in process of construction with walls built of these blocks. Most of the blocks are moulded so as to form a hollow wall, and are made to imitate natural stone.

Block construction has an advantage over poured walls, in that the blocks are thoroughly seasoned before they are set and hence no provision is required for expansion or contraction. The author believes that such walls are less liable to crack and will be more uniform in color and texture.

Concrete walls in various forms will undoubtedly be more extensively used during the coming decade.

Party Walls.—There is much diversity in building regulations regarding the thickness of party walls, although they all agree in that such walls should never be less than 12 ins. thick. About one-half of the laws require that party walls shall be of the same thickness as external walls; the remainder are about equally divided between making the party walls 4 ins. thicker or thinner than for independent side walls.

When the walls are proportioned by the rule previously given the author believes that the thickness of the party walls should be increased 4 ins. on each story. The floor load on party walls is obviously twice that on side walls, and the necessity for thorough fire protection is greater in the case of party walls than in other walls.

Curtain Walls.—In buildings of the skeleton type the outer masonry walls are usually supported either in every story or every other story by the steel framework, and carry nothing but their own weight. Such walls may, therefore, be considered as only one or two stories high, and are usually made only 12 ins. thick for the whole height of a twelve- or fifteen-story building.

"For skeleton construction, the Chicago ordinance allows veneer walls of 12 ins. thickness for any height within the maximum limit of 130 ft. The New York City building law requires the use of 12-in. curtain walls for 75 ft. of the uppermost height thereof, and 4 ins. additional thickness for every lower 60 ft. section down to the sidewalk level. But, on account of the severity of these requirements as applied to very high cage-construction buildings, permission is frequently given by the Board of Examiners, who are empowered to modify the building laws within certain limits, to reduce the above-mentioned thickness to 12 ins. and 16 ins. for buildings greatly exceeding 100 ft. in height. They have never, however, permitted a uniform thickness of 12 ins. for buildings over twelve stories in height." *

A few of the earlier tall buildings were built with self-sustaining walls, starting from the foundation, while columns were introduced merely to support the floors and to give additional stiffness.

"The 'World' Building, New York City, erected in 1890, is an extreme example of high-building construction, with self-sustaining walls. The main roof is 191 ft. above the street level, making thirteen main stories, above which is a dome containing six stories—in all, a height of 275 ft. above the street. The self-sustaining walls are built of sandstone, brick, and terra-cotta, the thickness increasing from 2 ft. at the top to as much as 11 ft. 4 ins. near the bottom, where the walls are offset to a concrete footing 15 ft. wide. The walls are vertical on the outside faces, the thickness being varied by inside offsets, so that the columns are recessed into the walls at the bottom, but emerge and are some distance clear of the walls at the top."—"Architectural Engineering," p. 148.

For a more extended discussion of curtain walls the reader is referred to Freitag's "Architectural Engineering," and to Birkmire's "Planning and Construction of High Office Buildings."

Durability of Iron Solidly Imbedded in Masonry.—I believe that, imbedded in lime-mortar at such depth as to protect it from the air, hoop-iron bond is indestructible.—M. C. Meigs, May 17, 1887.

Iron ties imbedded in cement concrete, even when under water, will not rust, and may be considered as imperishable, *provided* that the concrete does not crack so as to admit the water.

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* Architectural Engineering, p. 164.

Many of our most prominent engineers consider Portland cement a better preservative of iron or steel than any paint.

HYDRAULIC CEMENTS.

Two kinds of hydraulic cement are used in this country in laying up mason work, and in making concrete, cement floors, walks, etc., viz., natural-rock cement and Portland or artificial cement.

Natural-rock Cement.—These cements are made by burning and finely grinding a natural rock whose principal ingredients are carbonate of lime, carbonate of magnesia, and alumina (clay).

The principal localities in the United States where natural cements are made for shipment are: Rosendale, N. Y.; Akron, N. Y.; Louisville, Ky.; Utica, Ill.; La Salle, Ill.; Milwaukee, Wis.; Fort Scott, Kan.; Mankato, Minn.; and Cement, Ga.

Brands.—Natural cements are most generally known by the name of the locality from which the material is obtained, as, for instance, Rosendale Cement, Utica Cement, Louisville Cement, etc. Each manufacturer, however, has a registered brand or trade-mark for his product. The brands indicated below have probably the largest sale amongst natural cements.

"Brooklyn Bridge," "Hoffman," "Newark-Rosendale," all Rosendale cements. The "Utica," "Milwaukee," "Louisville," and "Fort Scott" cements are also extensively used in the Middle West, and are good cements.

PROPERTIES AND CHARACTERISTICS OF NATURAL CEMENTS.

Color.—Natural cements are not as uniform in color as the Portland cements, but vary from a light to a dark brown, according to the varying proportions of oxide of iron and impurities contained in the stone. "In Rosendale cement a light color indicates an inferior underburnt rock." Utica cement is almost a cream color.

Weight.—The Rosendale cements vary in weight from 49 to 56 pounds per cu. ft. Every barrel of the Newark-Rosendale and Brooklyn Bridge brands contains 300 lbs., net. Akron, Milwaukee, Utica, and Louisville cements weigh 265 lbs. per barrel, net.

Time of Setting.—The natural cements begin to set quicker than the Portland cements, generally within thirty minutes and always within an hour. Cement has set when it resists the impression of the finger-nail.

Lime paste added to cement mortar will delay the setting and make it work easier, but it also reduces its strength.

Lime should never be added when the mortar is to be used in a wet or damp place.

Strength.—A good natural cement should show a tensile strength, neat, of at least 80 lbs. per sq. in. when one week old, and 120 lbs. at the end of thirty days, or when mixed with one part sand, 40 lbs. at the end of one week and 70 lbs. at the end of a month. The best brands will give results 25% in excess of these figures.

The strength of 1 to 2 natural-cement mortar is about equal to Portland-cement mortar 1 to 4.

Proportions of Cement and Sand for Mortar and Concrete.—For mortar for stone rubble and ordinary brickwork one part of natural cement may be mixed with three parts of sand by measure.

For brick piers and first-class brickwork, not more than two parts sand to one of cement, by measure, should be used, and one or one and one-half parts of sand will make a still stronger mortar.

Any admixture of sand with cement reduces the strength.

For cement plastering, use equal parts of sand and cement.

For concrete, natural cements may be used in the proportion of one part cement to two parts of sand and four of gravel or stone chips.

Mortar that has set should not be retempered, but should be thrown away, as it will not take a true set a second time.

Natural-cement mortar possesses sufficient adhesion and crushing strength for any ordinary masonry, and when mixed in the proportion of 1 to 2 is probably just as good for masonry above ground and for ordinary foundations as Portland cement, but at the present price of Portland cement, it is not much, if any, cheaper than 1 to 3 Portland-cement mortar.

Effects of Freezing on Natural Cement.—It is commonly stated that natural cement should never be used in freezing weather, but an elaborate series of tests on frozen briquettes, published in the *Engineering Record* of Dec. 31, 1899, would

seem to show that Rosendale-cement mortar is not injured by freezing *in air*, but that it is not safe to let it freeze in water in less than two months.

ARTIFICIAL CEMENTS.

The artificial cements used in this country for laying up masonry, or in making concrete, are of three varieties, viz., true Portland cement, silica (sand) Portland cement, and Puzzolan (slag) cement.

Portland Cement.—The true Portland cements are made by thoroughly mixing together, in suitable proportions, clay and finely pulverized carbonate of lime (either chalk, marl, or compact limestone), burning the mixture in kilns at a high heat and then grinding the burnt product to fine powder.

"Pure Portland cement as known to-day by architects and engineers is strictly a mechanical mixture. Some manufacturers use as raw material clay and chalk, some marl and clay; others use argillaceous limestone rock properly dosed, while the first original Portland was made from mud dredged out of the river-beds of the lower Thames and the Medway, together with limestone."

True Portland cements are also now made which have slag for their hydraulic base, the "Universal" brand of Portland cement being a prominent example. Such cements differ in no way from other standard Portland cements, and are accepted in competition with them.

The chemical composition of a good Portland brand will be about as follows: Lime, 60.1; silica, 23.16; alumina, 8.5; ferric oxide, 5.3; with less than five parts of magnesia and sulphides.

Previous to the year 1872, all of the Portland cement used in this country was imported. In 1895, 2,300,000 lbs. of Portland cement was made in the United States, as compared with 13,500,000 in Germany and 8,300,000 in England. At the present time more than 90% of the Portland cement used in this country is of domestic production, and as American cements are fully equal in strength and durability to the imported cements, it will probably not be long before the importation of Portland cement, except a few brands for special purposes, will practically cease.

Silica-Portland Cement is a mixture of true Portland cement and siliceous sand ground together into an impalpable powder in a tube-mill.

A mixture of equal parts of sand and cement thus ground together possesses about the same strength as ordinary Portland cement alone.

A mixture of silica cement (one part cement and one part sand) with three parts unground sand has the same composition as one part cement and seven parts sand, but possessing the strength of a mixture of one part cement and three parts sand.*

The silica-cement process was first introduced into Denmark and has the special advantage of making mortar that is impermeable to moisture and able to resist the action of the elements.

During the years 1896–1902 a great many silica-Portland cement factories were erected abroad and several companies were formed in New York, Pennsylvania, and Illinois to manufacture it on a large scale. The author understands, however, that at least some of the factories have been abandoned, and that this material is now, 1905, used to a comparatively small extent, if at all.

Eight thousand barrels of silica cement were used in the foundations of the Cathedral of St. John the Divine, New York City.

Puzzolan Cement (sometimes called slag cement) is made from granulated slag of a certain composition, both physical and chemical, ground to exceeding fineness with quicklime which has been slaked with a solution of caustic soda. The product is a mechanical mixture of slag and slaked lime, no clinker being first produced as in the manufacture of true Portland cement. "Steel Puzzolan cement" is ground to a fineness of 96% through a 40,000-mesh sieve.

"Puzzolan cement made from slag is characterized physically by its light lilac color; the absence of grit attending fine grinding and the extreme subdivision of its slaked-lime element; its low specific gravity (2.6 to 2.8) compared with Portland (3 to 3.5); and by the intense bluish-green color in the fresh fracture after long submersion in water, due to the presence of sulphides, which color fades after exposure to dry air.

"Puzzolan cement properly made contains no free or anhydrous lime, stands storage well, does not warp or swell, but is

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* Addison H. Clark, in Architects' Handbook of Cements.

liable to fail from cracking and shrinking (at the surface only) in dry air.

"Mortars and concretes made from Puzzolan approximate in tensile strength similar mixtures of Portland cement, but their resistance to crushing is considerably less. On account of its extreme fine grinding Puzzolan often gives nearly as great tensile strength in 3 to 1 mixtures as neat.

"Puzzolan permanently assimilates but little water compared with Portland, its lime being already hydrated. It should be used in comparatively dry mixtures, well rammed.

"The cement is well adapted for use in sea-water, and generally in all positions where it will be constantly exposed to moisture, such as in foundations, sewers and drains, and underground works generally, and in the interior of heavy masses of masonry or concrete." *

It should not be used for work exposed to dry air or mechanical wear, as in floors and sidewalks, nor should it be used in connection with iron or steel, unless the metal is well protected by some coating.

Stainless Cements.—Mortar mixed with natural-rock cements, or ordinary Portland cements, are likely to produce stains in limestone and marble, and sometimes in granite.

There are a few Portland cements which do not cause stains, and if any cement at all is to be used in the mortar for setting or pointing, one of these brands should be specified.

Lime mortar does not stain the stones, but of course it does not make as strong a job as cement mortar.

Leading Brands of Portland Cement.—The following are among the leading brands of Portland cement now on the market.

American Portland: "Atlas," † "Alpha," "Dragon," "Lehigh," "Iron Clad," "Saylors," "Vulcanite" (Iola, Colorado Portland, Red Diamond ‡).

American Puzzolan: "Steel Puzzolan."

English Portland: "Brooks, Shoobridge & Co."

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* Report of Board of Engineer Officers, U. S. Army, on Testing Hydraulic Cements. 1901.

† The output of Atlas cement in 1901 was over 3,000,000 barrels.

‡ These cements are made in Kansas, Colorado, and Utah, respectively, and are extensively used in those States.

German Portland: "Alsen's," "Dyckerhoff," "Mannheimer," "Germania."

Stainless cements: "La Farge," French Portland; "Puzzolan," H. H. Meier & Co., Bremen; "Rhinoceros," American.

Cost of Portland Cement.—Portland cement can now be purchased in this country at prices from $1.25 to $2.50 per barrel, the former price being for 5,000-barrel lots at the factory. For a large order, in any of the Central States, cement can probably be obtained at $2.00 per barrel, delivered. The retail price for single barrels varies from about $2.25 to $2.75 per barrel.

In Germany the average price per barrel in the open market appears to be about $1.25.

PROPERTIES AND CHARACTERISTICS OF PORTLAND CEMENT.

Color.—Portland cement should be of a greenish gray color. Slag cements are of a light gray color.

Weight.—The weight of Portland cement, *loose,* varies from 77 to 95 lbs. per cu. ft., the average being from 85 to 90 lbs. A barrel of Portland cement is supposed to contain 3½ cu. ft. (packed), or about 380 lbs. net of cement. When put up in sacks, each sack is supposed to contain 95 lbs., or four sacks to the barrel.*

"Steel Puzzolan" (slag) cement weighs 330 lbs. net to the barrel, or 82½ lbs. per bag or sack.

Fineness.—Fineness in cement is a very important quality, as fine grinding increases the strength and sand-carrying capacity. The fineness of Portland cement when ready for the market should be such that not less than 95% will pass through a

*The actual weight per cubic foot and per bbl. of several brands of Portland cement, as tested by Chas. G. Reid, is shown by the following table:

Brand.	Volume per barrel in cubic feet loose measure	Weight per barrel in pounds.		Weight per cubic foot loose measure.
		Gross.	Net.	
Dyckerhoff	4.47	395	369.5	83
Atlas	4.45	401	381	85.5
Alpha	4.37	400.5	381	86.5
Meiers	4.84	375	353.5	73.5
Steel	**4.96**	**350**	**332.5**	**67.5**
Hilton	4.64	393	370.5	79.5

sieve of 2,500 meshes to the square inch. The U. S. Corps of Engineers requires that 92% shall pass a sieve of 10,000 meshes to the sq. inch. Some American cements will pass 99%. The finer a cement is ground the more bulky it becomes, and the less it weighs by measure.

Time of Setting.—A true Portland cement is classed as slow setting when a cake of neat cement takes over a half hour to harden. Sand retards setting, so that cement which when mixed neat would set in half an hour may not set for one or two days if mixed with large proportions of sand.

Strength.—The best test for the strength of Portland cement is the tensile strength of briquettes composed of 1 part cement and 3 parts standard sand.

Such briquettes should show a strength at seven days of from 100 to 140 lbs., at twenty-eight days of from 200 to 300 lbs., and at the end of one year of from 300 to 400 lbs.

Briquettes mixed neat (without sand) should break at seven days, at from 250 to 550 lbs.; at twenty-eight days, at from 400 to 800 lbs., and at the end of a year, at from 500 to 1,000 lbs. The author has made briquettes which gave a breaking strength of over 1,000 lbs. when twenty-eight days old. The briquettes should be kept in a damp box for the first twenty-four hours, and then in water.

The crushing resistance of Portland cement varies from 8 to 12 times the tensile strength, the average being 10 times the tensile strength.

The greater the increase per cent. between the seven-day and twenty-eight-day tests, the stronger and harder the cement is likely to become. This increase should be at least 25%.

Water Required in Mixing.—Good Portland cement requires but little water to make a good mortar. Neat cement will take 17% to 20% (by weight) of water, a quick-setting cement requiring more water than one that is slow setting.

If a greater quantity of water is required, it indicates the presence of an excess of free lime.

When sand is mixed with cement, in the proportion of 3 to 1, not more than 9% to 12½% (by weight) of water will be required.

Natural rock and slag cements require more water than do Portland cements.

Too much water "drowns" the cement, retards the setting, and weakens the mortar.

Cement can also be spoiled by a deficiency of water.

Portland-Cement Mortar.—For first-class mortar not more than 3 bbls of sand should be added to 1 bbl. of cement. For rubble stonework under ordinary conditions a mortar composed of 4 parts sand to 1 of cement will answer every purpose, and be much stronger than lime mortar.

For the top surface of floors and walks, from 1 to 1½ parts of sand may be mixed with 1 part cement.

1 to 3 Portland-cement mortar has about the same strength at the end of one year as 1 to 1 natural rock-cement mortar.

Mortar made with *fine* sand requires twice the quantity of cement to obtain a given strength as that made with coarse sand.

Effects of Freezing on Portland-Cement Mortar.—Numerous experiments and the experience of engineers bear out the assertions,

1st, that the mortar is considerably injured, but not totally, if frozen before it is set.

2d. That freezing only partially suspends chemical action in the setting of cement.

3d. It is not safe to allow a slow-setting cement mortar to freeze in less than four days after it has been placed, while a very quick-setting mortar may freeze in twelve hours without injury, provided the mortar is kept frozen until set.

4th. That Portland-cement mortar is injured more when it alternately freezes and thaws than when it remains frozen before it has set hard.*

5th. If salt is added to the water of mixture no bad effects will result from freezing. The rule for the proportion of salt used in the works at Woolwich Arsenal, is said to have been—"Dissolve one pound of rock salt in eighteen gallons of water when the temperature is at 32 degrees Fahr., and add three ounces of salt for every three degrees of lower temperature."

6th. Hot water hastens the setting of Portland-cement mortar.

7th. 2 lbs. carbonate of soda in 1 gal. of water boiled and mixed in mortar hastens the setting and protects from freezing.

Quantity of Mortar required for Masonry and Plastering.†—

"One barrel of Portland cement and three barrels of sand thoroughly and properly mixed will make 3⅓ bbls., or 12 cu. ft., of good strong mortar. This will be sufficient to lay up 1½ cu.

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* See *Engineering Record* for December 24 and 31, 1898.

† These figures can be considered as approximate only, as the amount of mortar will vary on different jobs.

yds. of rough stone, or about 750 bricks, with $\frac{1}{4}$ to $\frac{3}{8}$-in. joints, or cover 125 square feet of surface 1 in. thick, or 250 sq. ft. $\frac{1}{2}$ in. thick."

"One barrel of Rosendale cement and two barrels of lime, mixed with about half a barrel of water, will make 8 cu. ft. of mortar, sufficient to lay 522 common bricks, with $\frac{1}{4}$ to $\frac{3}{8}$ in. joint, or about 1 cu. yd. of rough rubble."

For the top coat of walks or floors.

1 bbl. of Portland cement and 1 of sand will cover 75 to 80 sq. ft., $\frac{1}{2}$ in. thick, or 50 to 56 sq. ft. $\frac{3}{4}$ in. thick.

1 bbl. of Portland cement and $1\frac{1}{2}$ bbls. of sand will cover 110 to 120 sq. ft. of floor $\frac{1}{2}$ in. thick, or 75 to 80 sq. ft. $\frac{3}{4}$ in. thick.

CONCRETE.

There is probably no material that is so enduring or better adapted for foundations, walks, and basement floors than cement concrete, and for a certain class of buildings it may be used with advantage for the walls, floors, and interior supports. In fact there are now probably one hundred buildings in this country in which all of the structural portions are formed of concrete, and the use of Portland-cement concrete for a great variety of purposes is rapidly extending, due largely to the reduced price of Portland-cement, and also to a better appreciation of its merits.

Concrete may be defined as an artificial rock, made by uniting sand, broken stone, gravel, fragments of brick, pottery, etc., by means of lime or cement.

Concrete made with lime, however, is not suitable for damp situations, and even when used for walls above ground it is much better to use either a "Portland" or "natural" cement for the uniting material; in fact lime is no longer used for this purpose.

Concrete made with good Portland cement, in proper proportions, becomes so hard and strong that when pieces of the concrete are broken the line of fracture will often be found to pass through the particles of stone, showing that the adhesion of the cement to the stone is greater than the strength of the stone.

For the aggregates no material is better than clean, freshly broken stone, in size about as large as a hen's egg. Granite probably makes the best aggregates, but other hard stones will answer for any ordinary concrete. Soft sandstones or "free-

stones" are not desirable. Pieces of hard brick or dense terra-cotta also make good aggregates.

Whatever material is used it is essential that it be free from dirt and that the particles be clean.*

Good clean coarse gravel is also extensively used for the mass of the concrete, and some architects and builders prefer it to broken stone, but as all gravel has more or less rounded and smooth surfaces, it would seem as though the cement must adhere more firmly to angular and broken surfaces.†

A certain proportion of clean coarse sand is also required to fill the voids between the particles of stone or gravel.

The best proportion of cement, sand, and aggregates will depend upon the kind and quality of the cement used, the character of the aggregates, and of the work.

Proportions.—The proportion of sand to aggregates should be such that the sand will just fill the voids in the aggregates. This will, of course, vary with the size of the aggregates and the coarseness of the sand. For stone broken to go through a $2\frac{1}{2}$-inch ring about one-half as much sand as stone is required, on an average, to fill the voids. After one batch of concrete has been deposited and rammed the inspector can generally tell by the appearance whether too much or too little sand has been used.

Natural-Cement Concrete.—For concrete foundations under buildings of moderate height, and for foundations for cement pavements, natural cement makes as strong a concrete as is required.

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* Mr. G. J. Griesenauer, cement tester for the Chicago, Milwaukee & St. Paul Ry., reports, in the *Engineering News* of April 16, 1903, a very interesting series of tests on the comparative strength of cement mortar made from Trorpedo sand and limestone and gravel screenings. These tests seem to show that limestone screenings make very much stronger mortar than sand, the increase in strength averaging about 115 per cent. for proportions of 1 to 3. Gravel screenings gave about the same strength as sand.

Mr. W. A. Rogers, formerly with the same railroad, in a very valuable paper on concrete, reports tests which seem to show that "a small amount of dirt in the sand is probably not seriously objectionable, if suitable in other ways."

† From some experiments carried out under the direction of Captain Wm. Black, of the United States Corps of Engineers, for the purpose of determining the influence of different aggregates on the strength of concrete, it would seem that the gravel makes a much weaker concrete at the start than stone, especially when natural cements are used, but that after a period of one year it will probably attain the same strength as that made of broken stone. (See *Engineering Record* for April 9, 1898.)

For the best brands of natural cements 1 part cement, 2 parts sand, and 4 parts gravel or broken stone should be used.

(This proportion was used in the foundations of the Brooklyn Bridge.)

Portland-Cement Concrete.—For concrete to be used under heavy buildings and under water Portland cement should be used.

For the best brands of cement 2 parts of cement to 5 of sand and 9 of broken stone will answer for almost any building construction. Much larger proportions of sand and aggregates than these are often used, but the author would not recommend a greater proportion than the above unless the quality of the cement is constantly tested and only the best used, and the concrete mixed under rigid inspection.

Manner of Mixing.— The most satisfactory method of mixing concrete by hand is to first prepare a tight floor of plank, or, better still, of sheet iron with the edges turned up about 2 ins., for mixing the materials on.

Upon this platform should first be spread the sand, and upon this the cement. The two should then be thoroughly and immediately mixed by means of shovels or hoes, and the broken stone or aggregates then dumped on top and the whole worked over dry with shovels, and then again worked over while water is added from a sprinkler on the end of a hose. After enough water has been added the mass should be worked over at least twice. Only as much water should be added as is necessary to enable the mortar to completely coat and cause to adhere all the particles of the aggregates, and so that when the concrete is tamped the water will just flush to the surface without quaking.

The water should be clean and at about the temperature of 65 degrees.

There are many machines for mixing mortar which for large quantities of concrete effect a material saving in the cost of mixing, and probably do the work more thoroughly and evenly. As soon as the concrete is mixed it should be wheeled to the trenches in barrows and deposited in layers not over 6 ins. thick and well tamped. After it is deposited concrete should be protected from drying too rapidly, as Portland cement reaches its maximum strength only when kept damp. If the concrete dries quickly it is also liable to crack from contraction, which in exposed work is likely to lead to its destruction by weathering.

Effect of Freezing on Concrete.—It would seem that

the general opinion among railway engineers is that when careful precautions are taken in laying Portland-cement concrete, freezing weather will not cause any trouble, while the effect of frost on concrete that has set either amounts to nothing or is confined to surface cracks.*

Cost of Concrete and Materials Required per Yard.—The quantities of cement, sand, and gravel required to make a yard (27 cu. ft.) of concrete will vary somewhat on different jobs. The values given in the following tables may be used as fair averages for making estimates.

QUANTITIES REQUIRED FOR 1 CUBIC YARD OF RAMMED CONCRETE. (Compiled by Edwin Thacher, C.E.)

Mixtures.			Stone.†			Gravel.‡		
Cement.	Sand.	Stone.	Cement, bbls.	Sand, cu. yds.	Stone, cu. yds.	Cement, bbls.	Sand, cu. yds.	Gravel, cu. yds.
1	1.0	2.0	2.63	0.40	0.80	2.30	0.35	0.74
1	1.0	3.0	2.10	0.32	0.96	1.89	0.29	0.86
1	1.5	3.0	1.90	0.43	0.87	1.71	0.39	0.78
1	1.5	4.0	1.61	0.37	0.98	1.46	0.33	0.88
1	2.0	3.0	1.73	0.53	0.79	1.54	0.47	0.73
1	2.0	4.0	1.48	0.45	0.90	1.34	0.41	0.81
1	2.0	5.0	1.29	0.39	0.98	1.17	0.36	0.89
1	2.5	4.0	1.38	0.53	0.84	1.24	0.47	0.75
1	2.5	5.0	1.21	0.46	0.92	1.10	0.42	0.83
1	2.5	6.0	1.07	0.41	0.98	0.98	0.37	0.89
1	3.0	6.0	0.91	0.42	0.97	0.84	0.38	0.89

ACTUAL VOLUME OF RAMMED CONCRETE RESULTING FROM DIFFERENT PROPORTIONS OF INGREDIENTS. (As determined by Messrs. A. W. Dow and W. J. Douglas.§)

Ingredients.				Proportions.	Quantity of Concrete.
Cement.	Sand.	Stone.	Gravel.		
1 bbl. =	9 cu. ft.	20¼ cu. ft.	0	1 : 2 : 5	21.4 cu. ft.
4½ cu. ft.	11¼ "	27 "	0	1 : 2½ : 6	27.66 "
or	11¼ "	13½ "	13½ cu. ft.	1 : 2½ : 3 : 3	27.66 "
378¼ lbs.	13.5 "		45 "	1 : 3 : 10	45 "

For sand and gravel mixed as it comes from the pit 125 yds. will make about 100 yds. of concrete.

At $2 a day for common labor the cost of mixing and depositing concrete, by hand, will vary from $1.25 to $1.50 a cu. yd.

On large jobs concrete can be mixed by machines, deposited and tamped by hand at from .75 to .90 per cu. yd.

* *Engineering Record*, October 20, 1900, also December 7, 1901.

† 2½ in. and under, dust screened out.

‡ ¾ in. and under.

§ See *Engineering News*, March 10, 1904, p. 226.

For small jobs where there are no special disadvantages $6 per cu. yd. without forms is perhaps a fair average price at the present time (1904), although with wages at $2 per day, and on large jobs, the work can be done at from $4.00 to $5.00 a yard.

On the Boston subway the prices for labor and materials were as follows, per cu. yd.*

Natural rock-cement concrete, $5.00 to $8.00.

Portland-cement concrete, $6.50 to $9.50.

The following exact figures, giving the cost per cu. yd. of concrete of an arched culvert of 26 ft. span, with wing walls and parapet, built near Pittsburg, Pa., in 1901, should be of value in estimating the cost of such work. The proportions were 1 to 8 and 1 to 10, and the mixing was done by hand.

For complete description of the work see the *Engineering Record* for April 12, 1902.

The finished structure contained 1,439 cu. yds. of concrete masonry, the total cost of which was $7,243.24.

The cost per cu. yd. of concrete for material and labor was as follows:

Material.

Coarse gravel, 19 cts. per ton, 1.03 tons	$0.19½	
Fine gravel, 21 cts. per ton, 0.40 ton	.08½	
Sand, 36 cts. per ton, 0.32 ton	.11½	
Cement, $1.60 per barrel	1.53½	
Lumber	.43	
Tools and other storehouse accounts	.07¾	
		$2.43¾

Labor.

Preparing site and cleaning up after completion of structure, 15.5 cts. per hour	$0.21	
Forms, 23 cts. per hour	.28	
Platforms and buildings, 23 cts. per hour	.05	
Changing trestle, including service of work train and steam-derrick car	.08½	
Excavation, foundations, 15.5 cts. per hour	.31	
Handling material, 15.5 cts. per hour	.03¾	
Mixing and laying concrete, 15.5 cts. per hour	1.44	
		$2.41¼

Total cost per cubic yard of concrete $4.85

Wages paid were as follows: Foreman mason in general charge, 40 cents per hour; laborers, 15 cents per hour; foreman, 25 cents per hour; carpenters, 22.5 to 25 cents.

* Addison H. Clark, in "Architects' Handbook of Cement."

DATA FOR ESTIMATING THE COST OF A CUBIC YARD OF CONCRETE OF VARIOUS PROPORTIONS. (As compiled by W. A. Rogers, C.E.)

Proportion of materials.	Cost of labor, mixing and placing per cubic yard.	Cost of forms per cubic yard (where forms are required).	Materials per cubic yard.		
			Cement.	Sand.	Broken stone.
1 part of natural cement to 1½ parts sand to 4 parts broken stone......	90 per cent of the amount paid per day for labor.	From 35 cts. to 85 cts. per cubic yard.	1½ bbls.	0.35 cu. yd.	0.95 cu. yd.
1 part Portland cement to 2 parts sand to 5 parts broken stone......	"	"	1.2 bbls.	"	"
1 part Portland cement to 3 parts sand to 7½ parts broken stone......	"	"	0.9 bbls.	"	"

EXAMPLES OF PORTLAND CEMENT CONCRETE.

Foundation of U. S. Naval Observatory, Georgetown, D. C.: 1 part cement, 2½ sand, 3 gravel, 5 broken stone. (1 barrel of cement, 380 lbs., made 1.18 yds. of concrete.)

Foundations of Cathedral of St. John the Divine, New York: 1 part Portland cement, 2 parts sand, 3 parts quartz gravel, 1½ to 2 ins. in diameter. (17,000 barrels of cement made 11,000 yards of concrete.)

Manhattan Life Insurance Building, New York, filling of caissons: 1 part Alsen Portland cement, 2 parts sand, 4 parts broken stone.

Filling of caissons, Johnston Building (15 stories), New York: 1 part Portland cement, 3 parts sand, 7 parts stone, finished on top for brickwork with 1 part cement and 3 parts gravel.

Prof. Baker states that the concrete foundations under the Washington Monument were made of 1 part Portland cement, 2 parts sand, 3 parts gravel, and 4 parts broken stone, and that this mixture stood, at six months old, a load of 2,000 lbs. per sq. in., or 144 tons per sq. ft. For the strength of concrete see Chapter V.

The weight of concrete varies from 130 to 140 lbs. per cu. ft., according to the material used, granite aggregates making naturally the heaviest concrete.

CHAPTER IV.

RETAINING WALLS—VAULT WALLS.

A Retaining Wall is a wall for sustaining a pressure of earth, sand, or other filling or backing deposited behind it after it is built, in distinction to a *brest* or *face* wall, which is a similar structure for preventing the fall of earth which is in its undisturbed natural position, but in which a vertical or inclined face has been excavated.

Fig. 1 gives an illustration of the two kinds of wall.

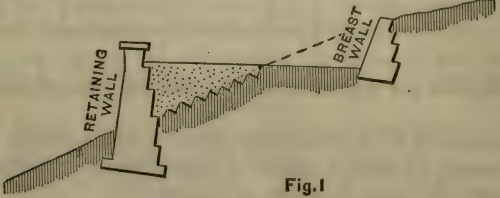

Fig. 1

Retaining Walls.—A great deal has been written upon the theory of retaining walls, and many theories have been given for computing the thrust which a bank of earth exerts against a retaining wall, and for determining the form of wall which affords the greatest resistance with the least amount of material.

There are so many conditions, however, upon which the thrust exerted by the backing depends,—such as the cohesion of the earth, the dryness of the material, the mode of backing up the wall, etc.,—that in practice it is impossible to determine the exact thrust which will be exerted against a wall of a given height.

It is therefore necessary, in designing retaining walls, to be guided by experience rather than by theory. As the theory of retaining walls is so vague and unsatisfactory, we shall not offer any in this work, but rather give such rules and cautions as have been established by practice and experience.

In designing a retaining wall there are two things to be considered,—the backing and the wall.

The tendency of the backing to slip is very much less when it is in a dry state than when it is filled with water, and hence every

precaution should be taken to secure good drainage. Besides surface drainage, there should be openings left in the wall for the water which may accumulate behind it to escape and run off.

The manner in which the material is filled against the wall also affects the stability of the backing. If the ground be made irregular, as in Fig. 1, and the earth well rammed in layers inclined *from* the wall, the pressure will be very trifling, provided that attention be paid to drainage. If, on the other hand, the earth be tipped, in the usual manner, in layers sloping *towards* the wall, the full pressure of the earth will be exerted against it, and it must be made of corresponding strength.

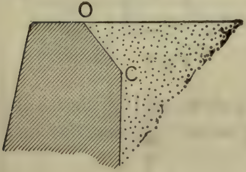

Fig.2

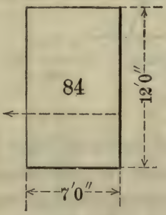

Fig. 3

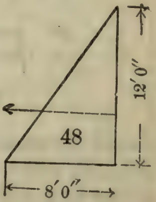

Fig.4

The Wall.—Retaining walls are generally built with a battering (sloping) face, as this is the strongest wall for a given amount of material; and, if the courses are inclined towards the back, the tendency to slide on each other will be overcome, and it will not be necessary to depend upon the adhesion of the mortar.

The importance of making the resistance independent of the adhesion of the mortar is obviously very great, as it would otherwise be necessary to delay backing up a wall until the mortar was thoroughly set, which might require several months.

The Back of the Wall should be left Rough.—In brickwork it would be well to let every third or fourth course below the frost-line project an inch or two. This increases the friction of the earth against the back and thus causes the resultant of the forces acting behind the wall to become more nearly vertical, and to fall farther within the base, giving increased stability. It also conduces to strength not to make each course of uniform height throughout the thickness of the wall, but to have some of the stones, especially near the back, sufficiently high to reach up through two or three courses. By this means the whole masonry becomes more effectually interlocked or bonded together as one

mass, and less liable to bulge. The courses of masonry are also often laid with their beds sloping in, as in Fig. 6, to overcome the tendency of the courses to slide on each other.

Where deep freezing occurs, the back of the wall should be

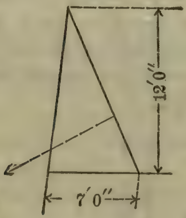

Fig. 5

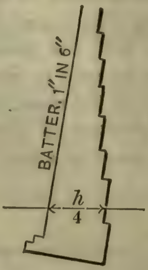

Fig. 6

sloped forwards for three or four feet below its top, as at *OC* (Fig. 2), which should be quite smooth, so as to lessen the hold of the frost and prevent displacement.

Figs. 3, 4, 5, and 6 show the relative sectional areas of walls of different shapes that would be required to resist the pressure of a bank of earth twelve feet high ("Art of Building," E. Dobson, p. 20). The first three examples are calculated to resist the maximum thrust of wet earth, while the last shows the modified form usually adopted in practice.

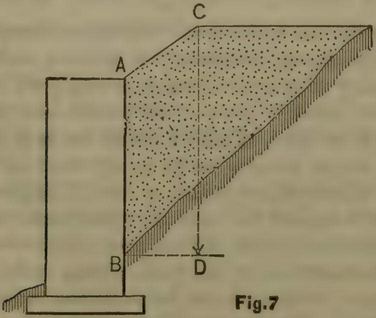

Fig. 7

Rules for the Thickness of the Wall.—As has been stated, the only practical rules for retaining walls which we have are empirical rules based upon experience and practice.

Trautwine, in his "Pocket-Book for Engineers," gives the following table for the thickness at the base of vertical retaining walls with a sand backing deposited in the usual manner.

The first column contains the vertical height *CD* (Fig. 7) of the earth as compared with the vertical height of the wall, *AB*; which latter is assumed to be 1, so that the table begins with backing of the same height as the wall. These vertical walls may be battered to any extent not exceeding an inch and a half to a foot, or 1 in 8, without affecting their stability, and without increasing the base.

Proportion of Retaining Walls.

(Thickness of wall in terms of the height, *A B*, Fig. 7).

Total height of the earth compared with the height of the wall above ground.	Wall of cut stone in mortar.	Good mortar, rubble, or brick.	Wall of good, dry rubble.
1	0.35	0.40	0.50
1.1	0.42	0.47	0.57
1.2	0.46	0.51	0.61
1.3	0.49	0.54	0.64
1.4	0.51	0.56	0.66
1.5	0.52	0.57	0.67
1.6	0.54	0.59	0.69
1.7	0.55	0.60	0.70
1.8	0.56	0.61	0.71
2	0.58	0.63	0.73
2.5	0.60	0.65	0.75
3	0.62	0.67	0.77
4	0.63	0.68	0.78
6	0.64	0.69	0.79

If the wall is built as in Fig. 8, with the ground practically level with the top, the top of the wall should be not less than 18 ins. thick, and the thickness at *a*, *a*, just above each step should be from one-third to two-fifths of the height from the top of the wall to that point. If the earth is banked above the top of the wall, the thicknesses should be increased as indicated by the table given above.

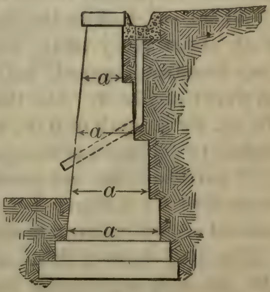

Fig. 8

If built upon ground that is affected by frost or surface water,

the footings should be carried sufficiently below the surface of the ground at the base to insure against heaving or settling.

Reinforced Concrete may be used to advantage in building retaining walls, and often at less expense than stone. Fig. 8A shows a wall suggested by the St. Louis Expanded Metal Fireproofing Co. which reduces the masonry to a minimum.

Brest Walls (from Dobson's "Art of Building").—Where the ground to be supported is firm, and the strata are horizontal, the office of a brest wall is more to protect than to sustain the earth. It should be borne in mind that a trifling force skilfully applied to unbroken ground will keep in its place a mass of material, which, if once allowed to move, would crush a heavy wall; and therefore great care should be taken not to expose the newly opened ground to the influence of air and wet for a moment longer than is requisite for sound work, and to avoid leaving the smallest space for motion between the back of the wall and the ground.

The strength of a brest wall must be proportionately increased when the strata to be supported incline towards the wall; where they incline from it, the wall need be little more than a thin facing to protect the ground from disintegration.

The preservation of the natural drainage is one of the most important points to be attended to in the erection of brest walls, as upon this their stability in a great measure depends. No rule can be given for the best manner of doing this: it must be a matter for attentive consideration in each particular case.

Vault Walls.—In large cities it is customary to utilize the space under the sidewalk for storage or other purposes. This necessitates a wall at the curb-line to sustain the street and also the weight of the sidewalk.

Where practicable the space should be divided by partition walls about every 10 ft., and when this is done the outer wall may be advantageously built of hard brick in the form of arches, as shown in Fig. 9. The thickness of the arch should be at least 16 ins. for a depth of 9 ft., and the "rise" of the arch from one-eighth to one-sixth of the span.

If partitions are not practicable each sidewalk beam may be supported by a heavy I-beam column, with either flat or segmental arches between of either brick or concrete.

Fig. 10 * shows a detail of the outer walls of the vault under the sidewalk around the Singer Building, New York; Mr. Er-

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* From *The Engineering Record* of February 26, 1898.

nest Flagg, architect. These walls consisted of a core formed by two-ring brick arches, with vertical axes built between the flanges of 8-inch vertical steel I beams spaced about 5 ft. apart and bedded at the bottom in a concrete footing. Their tops

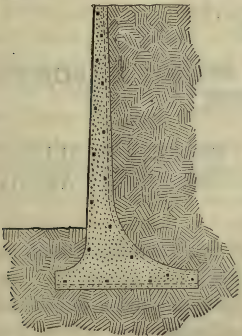

Fig. 8A

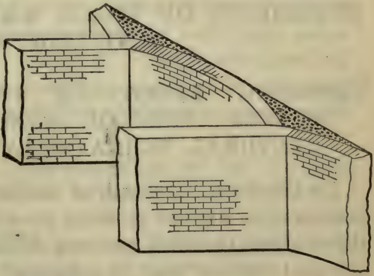

Fig. 9

were joined by 6-inch horizontal I beams and braced laterally by the sidewalk beams 5 ft. apart. The arches themselves

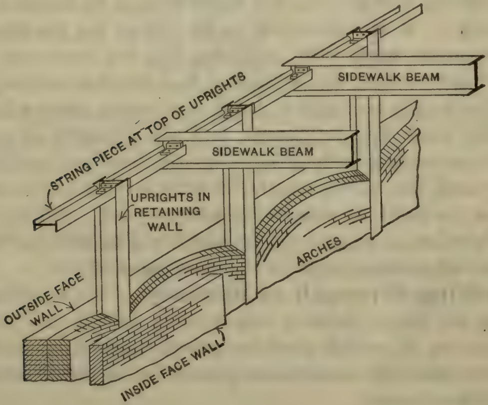

Fig. 10

were segmental with a rise of about 6 ins., and were built up solid against an 8-inch outside face wall. A 4-inch plain curtain wall was built inside against the flanges of the vertical beams inclosing segmental air chambers in front of each arch.

CHAPTER V.

STRENGTH OF BRICK AND STONE MASONRY AND CONCRETE.

CRUSHING RESISTANCE OF BRICK, BUILDING STONES, MORTAR, CONCRETES, AND ARCHITECTURAL TERRA-COTTA.

By the term "strength of masonry" is generally meant its resistance to a direct crushing force or load, and this is the only direct stress to which masonry should be subjected. Stone lintels and footings may be subjected to a transverse stress, but they can hardly be included in the term masonry, as they consist of single pieces. There is also more or less of a tendency to bend or split apart in brick walls and piers, as they are very high in proportion to their thicknesss, but this is a stress which cannot be accurately determined, and which should be avoided as much as possible. It is impossible to fix values for the strength of brick or stone work with anything like the exactness that we do for wood or steel, for the reason that there is not only a great variation in the strength of brick and stone, even when taken from the same kiln or quarry, but the strength of walls and piers is also very greatly affected by the kind and quality of the mortar used, the way in which the work is built and bonded, and whether the brick or stone is laid dry or wet. All that can be done, therefore, is to give values which will be safe for the different kinds of masonry built in the usual manner.

Working Strength of Masonry.—The building laws of most of the larger cities of this country specify the maximum loads per sq. ft. which shall be placed upon different kinds of masonry, which of course must govern the architect when building in those places.

When there is no restriction of this kind, Table I. will give a pretty good idea of the maximum loads which it is safe to put upon the different kinds of work indicated. Table II. gives the maximum safe loads as specified in the building laws of seven different cities, and in the latter part of the chapter is given records

of numerous tests made to determine the ultimate or breaking strength of various kinds of brick, building stones, mortars, and concretes, which are of value in determining the safe load for special cases.

In fixing the safe resistance of masonry from tests on the ultimate strength of work of the same kind, a factor of safety of at least 10 should be allowed for piers and 20 for arches.

The Chicago building ordinance fixes the maximum stress for dimension stone piers at one-thirtieth of the ultimate strength of the stone when the beds are dressed to a uniform bearing over their entire surface, and at one-fiftieth of the ultimate strength when the beds are not dressed.

When the stress exceeds one-seventieth of the ultimate strength the stones must be bedded in Portland-cement mortar.

TABLE I.—SAFE WORKING LOADS FOR MASONRY.

Brickwork in walls or piers.

	TONS PER SQUARE FOOT.	
	Eastern.	Western.
Red brick in lime mortar.	7	5
" hydraulic lime mortar.		6
" natural cement mortar, 1 to 3.	10	8
Arch or pressed brick in lime mortar.	8	6
" " " natural cement. . . .	12	9
" " " Portland cement . . .	15	12½

Piers exceeding in height six times their least dimensions should be increased 4 ins. in size for each additional 6 ft.

Stonework.

(Tons per square foot.)

Rubble walls, irregular stones.	3
" coursed, soft stone.	2½
" " hard stone.	5 to 16
Dimension stone, squared in cement:	
Sandstone and limestone.	10 to 20
Granite.	20 to 40
Dressed stone, with ⅜-inch dressed joints in cement:	
Granite.	60
Marble or limestone, best.	40
Sandstone.	30

Height of columns not to exceed eight times least diameter.

*Concrete.**

Portland cement, 1 to 8, 6 months, 10 tons, 1 year, 15 to 20 tons
Rosendale cement, 1 to 6, 6 months, 3 tons, 1 year, 5 to 8 tons

Hollow Tile.

(Safe loads per square inch of effective bearing parts.)

Hard fire-clay tiles. . . . . 80 lbs.†
" ordinary clay tiles. . . . . 60 "
Porous terra-cotta tiles. . . . . 40 "

Mortars.

(In ½-inch joints, 3 months old, tons per square foot.)

Portland cement, 1 to 4. 40
Rosendale cement, 1 to 3. 13
Lime mortar, best. . . . . 8 to 10
Best Portland cement, 1 to 2, in ¼-inch joints for bedding iron plates. 70

TABLE II.—COMPARISON OF BUILDING LAWS.

Materials.	Boston, 1897.	Buffalo, 1897.	New York, 1899.	Chicago, 1893.	St. Louis, 1897.	Philadelphia, 1899.	Denver, 1898.
	Allowable pressures in tons per sq. ft.						
Granite, cut.	60	....	72–172	....	....	....	40
Marble and limestone, cut.	40	....	50–165				
Sandstone, hard, cut.	30	....	28–115	....	....	....	12
Hard-burned brick in Port. cement	....	....	18	12½	15		
" " " in nat. cement .	15	9	15	9	....	15	9
" " " in cem't & lime.	12	....	11½	....	11½	12	
" " " in lime mortar .	8	6	8	6½	11	8	8
Pressed brick in Portland cement. .	....	12					
" " in natural cement. . .	....	9	......	....	....	....	12
Rubble stone in natural cement. . .	....	5	8	....	...	10	12
Dimension stone in foundations. . .	....	6	......	5–7	....	....	30
Portland cement concrete in foundations.	....	4	15	....	....	15	10
Natural cement concrete in foundations.	...	....	8	4	...	....	4

Brick Piers.—As a rule brickwork is subject to its full safe resistance only when used in piers, and in small sections of walls, under bearing-plates. In the latter case but a few

* See pp. 226–228.

† These loads are those allowed by the Chicago Building Ordinance.

courses receive the full load, and hence a greater unit stress may be allowed than for piers.

Values for computing the area of bearing plates are given in Chapter XIII.

Aside from the quality of the work and materials the two elements which most influence the strength of brick piers are the proportions of height to least horizontal dimensions and the method of bonding. When the height of a brick pier exceeds six times its least dimension the load per square foot should be reduced from the values given in Table I.

Formulas for the Safe Strength of Brick Piers exceeding six diameters in height.

From the records of numerous tests on the strength of brick piers, from some formulas published by Prof. Ira O. Baker in the *Brickbuilder* of April, 1898, and also from personal observation, the author has deduced the following formulas for the maximum working loads of first-class brickwork in piers whose height exceeds six times their least dimension.

Piers laid with rich lime mortar.

$$\text{Safe load per square inch} = 110 - 5\frac{H}{D}. \quad . \quad . \quad . \quad (1)$$

Piers laid with 1 *to* 2 *natural-cement mortar.*

$$\text{Safe load per square inch} = 140 - 5\tfrac{1}{2}\frac{H}{D}. \quad . \quad . \quad . \quad (2)$$

Piers laid with 1 *to* 3 *Portland-cement mortar.*

$$\text{Safe load per square inch} = 200 - 6\frac{H}{D}. \quad . \quad . \quad . \quad . \quad (3)$$

H representing the height in feet, and D the least horizontal dimension in feet.*

For a pier 20 ft. high and 2 ft. sq. these formulas will reduce the safe load to 4.3 tons per sq. ft. for lime mortar, 6.1 tons for natural cement mortar, and 10 tons for Portland cement mortar. No pier over 8 ft. high, should be less than 12″×12″, and when from 6 to 8 ft. in height they should be at least 8″×12″.

Brick piers intended to carry more than 50 per cent. of the safe loads given above should not be built in freezing weather nor with dry bricks. Lime mortar should not be used for

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* For piers faced with pressed brick, laid with ¼″ joint or less, and backed with common brick in lime mortar, only the dimensions of the backing should be considered in figuring the strength of the pier. If backing is laid in cement mortar, and face brick well tied to backing, the full section of pier may be considered.

For piers veneered with stone or terra-cotta, 4″ thick, only the strength of the backing should be considered.

building piers that will receive their full load within three months.

Effect of Bond on the Strength of Brick Work.—Brick piers, loaded to the point of destruction, always fail by the splitting and bulging out of the pier, and not by direct crushing of the brick or mortar, showing that the pier is weakest in the bond and in the tensile or transverse strength of the brick. It is very important therefore that the brickwork be well bonded, and all joints filled with mortar or grouted. The strength of a brick pier intended to carry an extreme load would probably be increased by bonding frequently with hoop iron in addition to the regular brick bond.*

Bond Stones in Piers.—Many competent architects and builders consider that the strength of a brick pier is increased by inserting bond stones, from 5 to 8 ins. in thickness and the full size of the pier, every 3 or 4 ft. in height.

The Chicago Building Ordinance requires that for all piers having a height four or more times their least dimension there shall be a bond stone at least 8 ins. thick for each distance in height equal to double that of the smallest dimension of such pier. The Building Laws for the City of New York require bond stones every thirty inches in height, and at least 4 ins. thick.

On the other hand, there are many first-class builders who consider that bond stones in a pier do more harm than good, and the author is of the opinion that this is generally the case. The Boston Building Law does not require bond stones. If bond stones are used, they should be bedded so as to bear rather more heavily on the inner portion of the pier than on the outer 4 ins., for unless this is done the outer shell will take most of the load, and will be likely to bulge away from the core.

Piers which support girders or columns should have a cap-stone or iron plate of sufficient strength to distribute the pressure over the entire cross-section of the pier.

Walls faced with Stone, Terra-cotta, or Cement Blocks.—Brick walls faced with blocks or ashlar of any material should always have the backing laid in cement—or cement and lime—mortar unless the backing is very thick, say 30 ins. or more. The aggregate thickness of the mortar joints in the backing is so much greater than in the facing, that any shrinkage or compression of the mortar tends to throw undue weight on the facing and to separate it from the backing.

* The manner in which brick piers fail is excellently shown by illustrations on page 79 of the *Brickbuilder* for May, 1896.

Veneering of any kind should be tied to the backing at least every 18 ins. in height.

The New York Building Code requires (Sections 28 and 29) that all bearing walls faced with brick laid in running bond, and all walls faced with stone ashlar less than 8 ins. thick, shall be of such thickness as to make the wall independent of the facing conform to that required for unfaced walls. Ashlar 8 ins. thick and bonded into the backing may be counted as part of the thickness of the wall.

Grouting.*—It is contended by persons having large experience in building that masonry carefully grouted, when the temperature is not lower than 40° Fahr., will give the most efficient result.

Many of the largest buildings in New York City have grouted walls.

The Mersey docks and warehouses at Liverpool, England, one of the greatest pieces of masonry in the world, were grouted throughout. It should be stated, however, that there are many engineers and others who do not believe in grouting, claiming that there is a tendency of the materials to separate and form layers.

Crushing Height of Brick and Stone.—If we assume the weight of brickwork to be 120 pounds per cubic foot, and that it would commence to crush under 700 pounds per square inch, then a wall of uniform thickness would have to be 840 ft. high before the bottom courses would commence to crush from the weight of the brickwork above.

Average sandstones at 145 pounds per cubic foot would require a column 5,950 ft. high to crush the bottom stones; an average granite at 165 pounds per cubic foot would require a column 10,470 ft. high. The Merchants' shot-tower at Baltimore is 246 ft. high, and its base sustains a pressure of six tons and a half (of 2,240 pounds) per square foot. The base of the granite pier of Saltash Bridge (by Brunel) of solid masonry to the height of 96 ft., and supporting the ends of two iron spans of 455 ft. each, sustains nine tons and a half per square foot.

Stone Piers.—Piers of good strong building stone laid in courses the full size of the pier, with the top and bottom courses bedded true and even, may be used to support very heavy loads. The height of such piers, however, should not exceed ten times the least dimension, and when it exceeds eight times the thickness, the safe load should be reduced.

The joints should not exceed ⅜ inch, and should be spread with

* See *American Architect*, July 21, 1887, p. 11.

1 to 2 Portland-cement mortar, kept back 1 inch from the face of the pier to prevent spalling of the edges.

A test of the strength of a limestone pier 12 ins. square is described under Tests on the Crushing Resistance of Stone in this chapter.

Rubble-work should not be used for piers whose height exceeds five times the least dimension, or in which the latter is less than 20 ins.

Records of Tests on the Crushing Resistance of Bricks.—Table III gives the results of some tests on brick, made under the direction of the author, in behalf of the Massachusetts Charitable Mechanics' Association.

TABLE III.—SHOWING THE ULTIMATE AND CRACKING STRENGTH OF THE BRICK, THE SIZE AND AREA OF FACE.

Name of brick.	Size.	Area of face in sq. ins.	Commenced to crack under lbs. per sq. in.	Net strength lbs. per sq. in.
Philadelphia Face Brick......	Whole brick	33.7	4,303	6,062
" " "	Whole brick	32.2	3,400	5,831
" " "	Whole brick	34.03	2,879	5,862
Average................	....	....	3,527	5,918
Cambridge Brick (Eastern)....	Half brick	10.89	3,670	9,825
" " "	Whole brick	25.77	7,760	12,941
" " "	Half brick	12.67	3,393	11,681
" " "	Half brick	13.43	3,797	14,296
Average................	....	....	4,655	12,186
Boston Terra-Cotta Co.'s Brick	Half brick	11.46	11,518	13,839
" " " "	Whole brick	25.60	8,593	11,406
" " " "	Whole brick	28.88	3,530	9,766
Average................	....	....	7,880	11,670
New England Pressed Brick...	Half brick	12.95	3,862	10,270
" " " ...	Half brick	13.2	8,180	13,530
" " " ...	Half brick	13.30	2,480	13,082
" " " ...	Half brick	13.45	4,535	13,085
Average................	....	....	4,764	12,490

The specimens were tested in the government testing-machine at Watertown, Mass., and great care was exercised to make the tests as perfect as possible. As the parallel plates between which the brick and stone were crushed are fixed in one position, it is necessary that the specimen tested should have perfectly parallel faces.

The bricks which were tested were rubbed on a revolving bed until the top and bottom faces were perfectly true and parallel.

The preparation of the bricks in this way required a great deal of time and expense; and it was so difficult to prepare some of the harder bricks that they had to be broken and only one-half of the brick prepared at a time.

The Philadelphia Brick used in these tests were obtained from a Boston dealer, and were fair samples of what is known in Boston as Philadelphia Face Brick. They were a very soft brick.

The Cambridge Brick were the common brick, such as is made around Boston. They are about the same as the Eastern Brick,

The Boston Terra-Cotta Company's Brick were manufactured of a rather fine clay, and were such as are often used for face brick.

The New England Pressed Brick were hydraulic-pressed brick, and were almost as hard as iron.

From tests made on the same machine by the United States Government in 1884, the average strength of three (M. W. Sands) Cambridge, Mass., face brick was 13,925 pounds, and of his common brick, 18,337 pounds per square inch, one brick developing the enormous strength of 22,351 pounds per square inch. This was a very hard-burnt brick.

Three brick of the Bay State (Mass.) manufacture showed an average strength of 11,400 pounds per square inch.

The New England brick are among the hardest and strongest brick in the country, those in many parts of the West not having one-fourth of the strength given above, so that in heavy buildings, where the strength of the brick to be used is not known by actual tests, it is advisable to have the brick tested.

Prof. Ira O. Baker, of the University of Illinois, reported some tests on Illinois brick, made on the 100,000 pound testing-machine at the university, in 1888–89, which give the crushing strength of soft brick at 674 pounds per square inch, average of three face brick, 3,070 pounds, and of four paving brick, 9,775 pounds.

In nearly all makes of brick it will be found that the face brick are not as strong as the common brick.

Tests of the Strength of Brick Piers Laid with Various Mortars.*—These tests were made for the purpose of testing the strength of brick piers laid up with different cement mortars, as compared with those laid up with ordinary mortar.

* Made under the direction of the author.

The brick used in the piers were procured at M. W. Sands's brickyard, Cambridge, Mass., and were good ordinary brick. They were from the same lot as the samples of common brick described above.

The piers were 8″ by 12″, and nine courses, or about 22½″ high, excepting the first, which was but eight courses high. They were built Nov. 29, 1881, in one of the storehouses at the United States Arsenal in Watertown, Mass. In order to have the two ends of the piers perfectly parallel surfaces, a coat about half an inch thick of pure Portland cement was put on the top of each pier and the foot was grouted in the same cement.

March 3, 1882, three months and five days later, the tops of the piers were dressed to plane surfaces at right angles to the sides of the piers. On attempting to dress the lower ends of the piers, the cement grout peeled off, and it was necessary to remove it entirely and put on a layer of cement similar to that on the top of the piers. This was allowed to harden for one month and sixteen days, when the piers were tested. At that time the piers were four months and twenty-six days old. As the piers were built in cold weather, the bricks were not wet.

The piers were built by a skilled bricklayer, and the mortars were mixed under his superintendence. The tests were made with the government testing-machine at the Arsenal.

The following table is arranged so as to show the result of these tests, and to afford a ready means of comparison of the strength of brickwork with different mortars. The piers generally failed by cracking longitudinally, and some of the brick were crushed.

8″×12″ pier. Common bricks laid in—	Ultimate strength of pier.	Pressure per sq. in. under which pier commenced to crack.	Ultimate strength per square inch.
	Lbs.	Lbs.	Lbs.
Lime mortar	150,000	833	1,562
Lime mortar, 3 parts; Portland cement, 1 part	290,000	1,875	3,020
Lime mortar, 3 parts; Newark and Rosendale cements, 1 part	245,000	1,354	2,552
Lime mortar, 3 parts; Roman cement, 1 part	195,000	1,041	2,030
Portland cement, 1 part; sand, 2 parts	240,000	1,302	2,500
Newark and Rosendale cements, 1 part; sand, 2 parts	205,000	708	2,135
Roman cement, 1 part; sand, 2 parts	185,000	1,770	1,927

The Portland cement used in these tests was made by Brooks, Shoobridge & Co., of England.

As the actual strength of brick piers is a very important consideration in building construction, some tests, made by the United States Government at Watertown, Mass., and contained in the report of the tests made on the Government testing-machine for the year 1884, are given as being of much value.

Three kinds of brick were represented in the construction of the piers, and mortars of different composition, ranging in strength from lime mortar to neat Portland cement. The piers ranged in cross-section dimensions from 8″×8″ to 16″×16″, and in height from 16 ins. to 10 ft.

The piers were tested at the age of from 18 to 24 months.

Table IV gives the results obtained and memoranda regarding the size and character of the piers.

Test of Mortar Cubes.—Table V shows the crushing strength of 6″ cubes of mortar made by the United States Government at Watertown, Mass., in the year 1884.

The mortar cubes were allowed to season in the open air a period of fourteen and a half months, when they were tested.

The age of the plaster cube was four months. It should be noticed that, while the cubes of Rosendale cement and lime mortar showed a greater strength than when sand alone was mixed with the cement, with the cubes of Portland cement and lime mortar the reverse was the case, differing from the result obtained by the author. This shows the necessity of a number and variety of tests.

Tests of the Crushing Resistance of Various Building Stones.

Sandstones.

*Long Meadow (Mass.) Stone.**— Reddish-brown sandstone, two blocks about 4″×4″ in cross-section and 8″ high.

Block No. 1 commenced to crack at 10,333 lbs. per square inch, and flew from the machine in fragments at 13,596 lbs. per square inch.

Block No. 2 commenced to crack at 3,012 lbs. per square inch and failed completely at 9,121 lbs. per square inch.

* These tests made with U. S. testing-machine at Watertown Arsenal, Mass.

TABLE IV.—TABULATED RESULTS OF THE ACTUAL CRUSHING STRENGTH OF BRICK PIERS.

BUILT OF FACE-BRICKS (M. W. SANDS, CAMBRIDGE, MASS.).

No. of test.	Nominal dimensions. Height.		Cross-section.		Composition of mortar.	Weight per cubic foot.	Sectional area.	First crack.	Ultimate strength. Total.	Per sq. in.	Per sq. ft.	% of single brick.
	Ft.	In.	In.	In.		Lbs.		Lbs.	Lbs.	Lbs.	Tons.	
11	1	4	8	8	1 lime, 3 sand	137.4	57.00	85,000	143,600	2,520	181.4	18.1
320	6	8	8	8	1 " , 3 "	133.5	57.76	50,000	108,400	1,877	135.1	13.5
12	1	4	8	8	1 Portland cement, 2 sand	136.3	57.76	200,000	218,100	3,776	271.8	27.1
321	6	8	8	8	1 " " 2 "	133.5	57.76	85,000	129,900	2,249	161.9	16.2
283	2	0	12	12	1 lime, 3 sand	....	132.25	140,000	257,100	1,940	139.7	13.9
284[1]	2	0	12	12	1 " 3 "	....	113.76	90,000	226,100	1,990	143.3	14.3
332	10	0	12	12	1 " 3 "	131.7	132.25	70,000	199,800	1,511	108.8	10.9
334[2]	10	0	12	12	1 " 3 "	125.0	115.44	100,000	208,600	1,807	130.1	13.0
286	2	0	12	12	1 Portland cement, 2 sand	....	132.25	200,000	486,000	3,670	264.2	26.4
326	10	0	12	12	1 " " 2 "	132.2	132.25	200,000	298,000	2,253	162.2	16.2

BUILT OF COMMON BRICKS (M. W. SANDS).

No. of test.	Height Ft.	Height In.	Cross-section In.	Cross-section In.	Composition of mortar.	Weight per cubic foot.	Sectional area.	First crack.	Total.	Per sq. in.	Per sq. ft.	% of single brick.
10	1	4	8	8	1 lime, 3 sand	135.6	60.80	66,000	148,800	2,440	175.6	13.3
12½	6	8	8	8	1 " 3 "	133.6	62.40	....	96,100	1,540	110.8	8.4
281	2	0	12	12	1 " 3 "	....	138.06	75,000	296,400	2,150	154.8	11.7
282[3]	2	0	12	12	1 " 3 "	....	119.58	120,000	244,600	2,050	147.6	11.2
331	9	9	12	12	1 " 3 "	131.5	138.06	70,000	154,300	1,118	80.5	6.1
330[4]	10	0	12	12	1 " 3 "	136.0	115.50	70,000	183,300	1,587	114.3	8.6
329	10	0	12	12	1 Portland cement 2 sand	131.0	138.06	....	276,600	2,003	144.2	10.9
387	2	8	16	16	1 " " 2 "	....	256.00	460,000	696,000	2,720	195.8	14.8
328	10	0	16	16	1 " " 2 "	....	256.00	340,000	483,100	1,887	135.8	10.3

[1] Has hollow core 4.25″×4.25″. [2] Has hollow core 4.1″×4.1″. [3] Has hollow core 4.5″×4.5″. [4] Has hollow core 4.75″×4.75″.

BUILT OF COMMON BRICKS (BAY STATE).

285	2	0	12	12	1 lime, 3 sand	....	146.41	95,000	201,000	1,370	98.6	12.0
288	6	0	12	12	1 " 3 "	....	144.00	70,000	163,200	1,133	81.6	9.9
289	6	0	12	12	1 " 3 "	119.7	144.00	100,000	174,300	1,210	87.1	10.6
291[5]	6	0	12	12	1 " 3 "	118.2	144.00	80,000	191,600	1,331	95.8	11.7
292[6]	6	0	12	12	1 " 3 "	118.1	156.25	110,000	189,200	1,211	87.2	10.6
325	7	10	12	12	1 " 3 "	120	144.00	100,000	169,100	1,174	84.6	10.3
327	10	0	12	12	1 " 3 "	118	144.00	90,000	133,100	924	66.6	8.1
335	10	0	8	12	1 " 3 "	107.0	96.00	35,000	90,200	940	67.7	8.2
333	10	0	12	16	1 " 3 "	118.7	192.00	80,000	148,500	773	55.7	6.8
301	6	0	12	12	1 Rosendale cement, 2 lime mortar	120.6	144.00	160,000	237,000	1,646	118.5	14.4
293	6	0	12	12	1 " " 2 sand	123.0	144.00	260,000	284,000	1,972	142.0	17.3
300	6	0	12	12	1 Portland cement, 2 lime mortar	120.3	144.00	150,000	203,200	1,411	101.6	12.4
294	6	0	12	12	1 " " 2 sand	119.7	144.00	220,000	258,000	1,792	129.0	15.7
290	6	0	12	12	Neat Portland cement	126.6	144.00	280,000	342,000	2,375	171.0	20.8

[5] Joints broken every 6 courses. [6] Bricks laid on edge.

TABLE IV$_A$.—TESTS OF BRICK PIERS—McGILL UNIVERSITY LABORATORIES, MARCH, 1897.

Dimensions of pier.	Mortar.	Brick.	Crushing strength, lbs. per square inch.		Age.
			At first crack.	Maximum load.	
8.1 ins. × 8.1 ins., 11.6 ins. high; joints ⅛ in. thick.	1 Canadian Portland, 3 sand.	Ordin'ry well-burnt flat brick	822	1,234	3 weeks.
8.1 ins. × 8.1 ins., 11.6 ins. high; joints ⅛ in. thick.	1 German Portland, 3 sand.	Do.	990	1,230	"
8.2 ins. × 8.3 ins., 10.5 ins. high; joints ½ in. thick.	1 English Portland, 3 sand.	La Prairie pressed keyed on one side	1,130	1,524	"
8.4 ins. × 8.4 ins., 10.75 ins. high; joints ¼ in thick.	1 Belgian Portland, 3 sand.	Do.	1,204	1,985	"

TABLE V.—TABULATED RESULTS, 6″ MORTAR CUBES.

CRUSHING STRENGTH.

No. of test.	Composition.	First crack.	Ultimate strength per sq. in.	Weight per cu. ft.
		Lbs.	Lbs.	Lbs.
3a	1 part lime, 3 parts sand	......	135	112
3b	1 " " 3 " "	......	119	111
3c	1 " " 3 " "	......	118	106
4a	1 part Portland cement, 2 parts sand	......	560	116
4b	1 " " " 2 " "	......	696	120
4c	1 " " " 2 " "	11,500	383	115
5a	1 part Rosendale cement, 2 parts sand	......	156	111
5b	1 " " " 2 " "	......	186	109
5c	1 " " " 2 " "	4,500	143	107
6a	Neat Portland cement	......	2,673	126
6b	" " "	95,000	3,548	129
6c	" " "	......	4,227	135
7a	Neat Rosendale cement	11,000	421	94
7b	" " "	19,000	615	99
7c	" " "	19,200	526	97
8a	1 part Portland cement, 2 parts lime mortar[1]	......	204	109
8b	1 " " " 2 " " "	......	198	110
8c	1 " " " 2 " " "	......	175	103
9a	1 part Rosendale cement 2 parts lime mortar[1]	......	194	105
9b	1 " " " 2 " " "	......	193	106
9c	1 " " " 2 " " "	......	162	105
	Plaster-of-Paris	......	1,981	74

[1] Lime mortar, 1 part lime, 3 parts sand.

*Sandstone from Norcross Bros., Quarries, East Long Meadow, Mass.—Soft Saulsbury.** Block No. 1, 4″×4″×8″ high, commenced to crack at 8,250 lbs. and failed at 8,812 lbs. per square inch.

Block No. 2, 4″×4″×8″ high, commenced to crack at 6,500 lbs. and failed at 8,092 lbs. per square inch.

*Hard Saulsbury.** Block No. 1, 4″×4″×8″ high (about), commenced to crack at 12,716 lbs. and failed at 13,520 lbs. per square inch.

Block No. 2, same size as No. 1, commenced to crack at 13,953 lbs. and failed at 14650 lbs. per square inch.

*Kibbe Stone.** Block No. 1, 6″×6″×6″, commenced to crack at 12,590 lbs. and failed at 12,619 lbs. per square inch.

* These tests made with U. S. testing-machine at Watertown Arsenal, Mass.

Block No. 2, same size as No. 1, commenced to crack at 12,185 lbs. and failed at 12,874 lbs. per square inch.

Brown Stone from the Shaler & Hall Quarry Co., Portland, Conn.*

Dimensions.			Sectional area, sq. ins.	First crack, lbs.	Ultimate strength, lbs. per sq. in.	Classification.
Height, ins.	Compressed surface, ins.					
2.50	2.50	2.45	6.13	84,800	13,980	1st quality
2.50	2.48	2.47	6.13	81,700	13,330	1st "
2.98	3.00	2.95	8.85	123,200	13,920	2d "
2.95	2.98	2.97	8.85	122,000	15,020	3d "
2.51	2.55	2.53	6.45	63,850	9,900	Bridge
2.48	2.48	2.52	6.25	58,340	9,330	Bridge

Brown Stone from the Middlesex Quarry Co., Portland, Conn.† Four nearly cubical blocks, about 1½″ square. Pressure per square inch at time of failure: No. 1, 10,928 lbs.; No. 2, 10,322 lbs.; No. 3, 8,252 lbs., and No. 4, 6,322 lbs.

Red Sandstone † *from Greenlee & Son's Quarries at Manitou, Colo.* One specimen failed at 11,000 lbs. per sq. inch; weight 140 lbs. per cu. ft.

Light-red Laminated Sandstone,‡ *from St. Vrain Cañon, Colo.* (a very hard stone, excellent for walks and foundations). Crushing strength on bed 11,505 lbs. per square inch; weight 150 lbs. per cubic foot.

Gray Sandstone ‡ (*free-working*) *from Trinidad, Colo.* Crushing strength 10,000 lbs. per square inch; weight 145 lbs. per cubic foot.

Gray Sandstone ‡ *from Fort Collins, Colo.* (laminated and similar in quality to the St. Vrain Stone). Crushing strength on bed 11,700 lbs. per square inch; weight 140 lbs. per cubic foot (one ton of this stone measures just a perch in the wall).

Granite.

Red Granite ‡ *from Platte Cañon, Colo.* Crushing strength per square inch 14,600 lbs.; weight per cubic foot 164 lbs.

* From tests made by Colt's Patent Fire-arms Manufacturing Co.

† These tests made with U. S. testing-machine at Watertown Arsenal, Mass.

‡ From tests made for the Board of Capitol Managers (of Colorado) by State Engineer E. S. Nettleton, in 1885, on two-inch cubes.

Lava Stone from the Kerr Quarries, near Salida, Colo. Four cubical blocks.*

Dimensions.			Sectional area, sq. ins.	First crack, lbs.	Ultimate strength,	
Height, ins.	Compressed surface, ins.				Lbs.	Lbs. per sq. in.
4.00	4.00	4.00	16.00	165,900	165,000	10,369
4.00	4.00	4.00	16.00	174,100	174,100	10,881
2.00	2.00	1.99	3.98	36,400	37,100	9,322
1.99	1.99	1.99	3.96	38,200	38,200	9,646

Lava Stone,† Curry's Quarry, Douglas County. Crushing strength, 10,675 lbs. per square inch; weight, 119 lbs. per cubic foot. (Experience has shown that this stone is not suitable for piers, or where any great strength is required, as it cracks very easily.)

Marble.

White marble quarried at Sutherland Falls, Vermont. Two cubical blocks about 6 ins. square.*

Block No. 1 commenced to crack at 9,750 lbs. per square inch and failed suddenly at 11,250 lbs. per square inch.

Block No. 2 did not crack until it suddenly gave way at 10,243 lbs. per square inch.

Test of a Limestone Pier.—A pier of Lemont limestone 1 ft. sq. in cross-section and 9 ft. high, composed of 7 stones with bearing surfaces planed perfectly true and parallel to natural bed and the joints washed with a thin grout of the best English Portland cement, was tested at the Watertown Arsenal for Gen. Wm. Sooy Smith, and only commenced to crack when the full power of the machine, 400 tons, was exerted.

Crushing Strength of Concrete.—Tests for crushing strength made on 6-in. cubes of concrete, made of one part silica Portland cement (1.1), two parts sand, and three parts gravel. The concrete was taken from the bucket just as it was ready to be laid in the foundations of the Cathedral of St. John the Divine.

Each result is the average of the crushing strengths of four sep-

* Tested at U. S. Arsenal, Watertown, Mass.

† From tests made by Denver Society of Civil Engineers in 1884, also on two-inch cubes.

arate cubes, made under exactly the same conditions at different periods:

7 days old crushed	at 77,162 lbs.	or 2,143	lbs. per sq. in.
14 " " "	" 83,225 "	" 2,312	" " " "
30 " " "	" 92,465 "	" 2,568	" " " "

The following table gives the crushing strength of 12-in. cubes of concrete prepared and tested by the Engineering Department of the District of Columbia in the years 1896 and 1897, the loads being in pounds *per square foot.* After the tests were completed it was found that the machine used gave results 8% too high, and the figures have not been corrected.

The figures for 1-year cubes are averages of 5 tests, all others give the mean of two tests.

A more complete record of the tests may be found in the *Engineering Record* for April 9, 1898.*

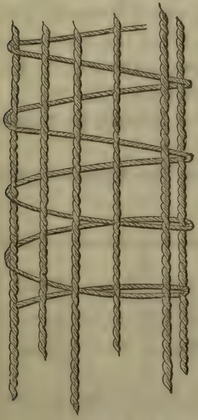
No.	Composition of concretes by volume.	10 days.	45 days.	3 mos.	6 mos.	1 year.
	1 part natural cement, 2 parts sand.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
1	6 parts average concrete stone.	32,900	77,687	54,022	114,412	131,700
2	3 parts average concrete stone, 3 parts gravel..	15,500	52,362	85,315	90,965	121,100
3	4 parts average concrete stone, 2 parts gravel.	....	....	....	....	131,700
4	6 parts (¾ average concrete stone, ¼ granolithic).	....	....	....	....	115,200
5	6 parts average gravel. ..	12,500	60,652	51,980	49,437	109,900
6	6 parts coarse concrete stone (no fine)	....	....	85,880	....	119,300
	1 part (Atlas) Portland cement, 2 parts sand.					
7	6 parts average concrete stone.	130,750	†343,520 172,325	324,875	361,600	440,040
8	3 parts average concrete stone, 3 parts gravel..	136,750	266,962	....	298,037	396,200
9	4 parts average concrete stone, 2 parts gravel..	....	....	....	....	408,300
10	6 parts (¾ average concrete stone, ¼ granolithic).	....	....	....	....	388,700
11	6 parts average gravel ..	99,900	234,475	385,612	265,550	406,700
12	6 parts coarse concrete stone (no fine).	....	....	234,475	220,350	266,300

* Valuable data on the crushing strength of concretes of different proportions may be found in the *Engineering News* of Feb. 4, 1904, p 114.

† Owing to the great variation in the strength of the two cubes, the results for both are given.

The average crushing strength of four 12-inch cubes of concrete, tested at the U. S. Arsenal, Watertown, Mass., for the city of Cleveland, Ohio, was 4,286 lbs. per sq. inch. The cubes were composed of 1 part Vulcanite Portland cement, 2 parts lake sand and 4 parts of broken limestone, and were 85 days old when tested.

Strength of "Hooped" Concrete Columns.—In 1892, M. Considère conducted a series of experiments at the laboratory of l'École des Ponts et Chaussées, which demonstrated conclusively that the resistance of concrete columns may be very greatly increased by winding spirally with wire and then plastering with cement. Crushing resistances of such columns as high as 9,150 lbs. per sq. inch on full section, and 12,700 lbs. per sq. inch on core section, were obtained. M. Considère considers that working values may be allowed for such columns of from 950 to 2,150 lbs. per sq. inch, according to the way in which the columns are made and reinforced. See the *Engineering Record* of Jan. 3, 10, 17 and 24, 1903.



Re-inforcing Skeleton for Concrete Columns.

The accompanying engraving shows the method of reinforcing concrete columns, employed by the Ransome Companies. With this reinforcement it is customary to allow a safe stress in the column of 800 to 1000 lbs. to the square inch.

Architectural Terra-Cotta—Weight and Strength.

The lightness of terra-cotta, combined with its enormous resisting strength, and taken in connection also with its durability and absolute indestructibility by fire, water, frost, etc., renders it specially desirable for use in the construction of all large edifices.

Terra-cotta for building purposes, whether plain or ornamental, is generally made of hollow blocks formed with webs inside, so as to give extra strength and keep the work true while drying. This is necessitated because good, well-burned terra-cotta cannot safely be made more than about 1¼ inches in thickness, whereas, when required to bond with brickwork, it must be

at least four inches thick. When extra strength is needed, these hollow spaces are filled with concrete or brickwork, which greatly increases the crushing strength of terra-cotta, although alone it is able to bear a very heavy weight. "A solid block of terra-cotta of one foot cube has borne a crushing strain of 500 tons and over."

TABLE VI.—CRUSHING RESISTANCE OF BRICK, STONE, AND CONCRETES. (PRESSURE AT RIGHT ANGLES TO BED.) *

		Pounds per sq. in.
Brick: Common, Massachusetts		10,000
Common, St. Louis		6,417
Common, Washington, D. C.		7,370
Paving, Illinois	6,000 to	13,000
Granites: Blue, Fox Island, Me.		14,875
Gray, Vinal Haven, Me.	13,000 to	18,000
Westerly, R. I.		15,000
Rockport and Quincy, Mass.		17,750
Milford, Conn.		22,600
Staten Island, N. Y.		22,250
East St. Cloud, Minn.		28,000
Gunnison, Colo.		13,000
Red, Platte Cañon, Colo.		14,600
Limestones: Glens Falls, N. Y.		11,475
Joliet, Ill.		12,775
Bedford, Ind.	6,000 to	10,000
Salem, Ind.		8,625
Red Wing, Minn.		23,000
Stillwater, Minn.		10,750
Sandstones: Dorchester, N. B. (brown)		9,150
Mary's Point, N. B. (fine grain, dark brown)		7,700
Connecticut brown stone,† on bed	7,000 to	13,000
Longmeadow, Mass. (reddish brown)	7,000 to	14,000
" " average, for good quality		12,000
Little Falls, N. Y.		9,850
Medina, N. Y.		17,000
Potsdam, N. Y. (red)	18,000 to	42,000
Cleveland, Ohio		6,800
North Amherst, Ohio		6,212
Berea, Ohio	8,000 to	10,000
Hummelstown, Pa.		12,810
Fond du Lac, Minn.		8,750
Fond du Lac, Wis.		6,237
Manitou, Colo. (light red)	6,000 to	11,000
St. Vrain, Colo. (hard laminated)		11,505
Marbles: Lee, Mass		22,900
Rutland, Vt.		10,746
Montgomery Co., Pa.		10,000
Colton, Cal.		17,783
Italy		12,156
Flagging: North River, N. Y.		13,425

* For more complete tables of the strength, weight, and composition of building stones, see *Building Construction and Superintendence*, Part I.

† This stone should not be set *on edge*.

Some exhaustive experiments made by the Royal Institute of British Architects give the following results as the crushing strength of terra-cotta blocks:

	Crushing wt. per cu. ft.
1. Solid block of terra-cotta.	523 tons
2. Hollow block of terra-cotta, unfilled.	186 tons
3. Hollow block of terra-cotta, slightly made and unfilled	80 "

Tests of terra-cotta manufactured by a New York Company, which were made at the Stevens Institute of Technology in April, 1888, gave the following results:

	Crushing wt. per cu. in.		Crushing wt. per cu. ft.
Terra-cotta block, 2-inch square, red.	6,840 lbs.	or	492 tons
Terra-cotta block, 2-inch square, buff.	6,236 "	"	449 "
Terra-cotta block, 2-inch square, gray.	5,126 "	"	369 "

From these results, the writer would place the safe working strength of terra-cotta blocks in the wall at 5 tons per square foot when *unfilled*, and 10 tons per square foot when *filled solid* with brickwork or concrete.

Strength of Terra-Cotta Brackets or Consoles.—A cornice modillion made by the Northwestern Terra-Cotta Company, 11½ ins. high at the wall line, 8 ins. wide on face, with a projection of 2 feet, was built into a wall and the upper surface loaded with pig iron to the extent of two tons without effect.

Another bracket, 5½ ins. high, 6 ins. wide, and 14 ins. projection made in the East, broke at wall line under 2,650 lbs., while a duplicate of it sustained 2,400 lbs. for one month without breaking. (See "The Brickbuilder," Vol. 7, p. 142.)

The *weight* of terra-cotta in *solid blocks* is 122 pounds. When made in hollow blocks 1½ inches thick the weight varies from 65 to 85 pounds per cubic foot, the smaller pieces weighing the most. For pieces 12″×18″ or larger on the face, 70 pounds per cubic foot will probably be a fair average.

For the exterior facing of fireproof buildings, terra-cotta is now considered as the most suitable material available.

CHAPTER VI.

COMPOSITION AND RESOLUTION OF FORCES—CENTRE OF GRAVITY.

Let us imagine a round ball placed on a plane surface at A (Fig. 1), the surface being perfectly level, so that the ball will have no tendency to move until some force is imparted to it. If, now, we impart a force, P, to the ball in the direction indicated by the arrow, the ball will move off in the same direction. If, instead of imparting only one force, we impart two forces, P and P_1 to the ball, it will not move in the direction of either of the forces, but will move off in the direction of the resultant of these forces, or in the direction Ab in the figure. If the magnitude of the forces P and P_1 is indicated by the length of the lines, then, if we complete the parallelogram $ABCD$, the diagonal DA will represent the direction and magnitude of a force which will have the same effect on the ball as the two forces P_1 and P. If, in addition to the two forces P_1 and P we now apply a third force, P_2, the ball will move in the direction of the resultant of all three forces, which can be obtained by completing the parallelogram $ADEF$, formed by the resultant DA and the third force P_2. The diagonal R of this second parallelogram will be the resultant of all three of the forces, and the ball will move in the direction Ae. In the same way we could find the resultant of any number of forces.

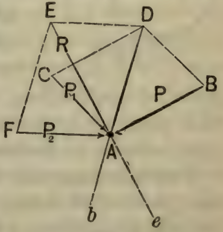


Fig. 1

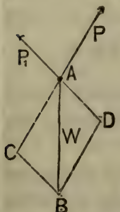


Fig. 2

Again, suppose we have a ball suspended in the air whose weight is indicated by the line W (Fig. 2). Now, we do not wish to suspend this ball by a vertical line above it, but by two inclined lines or forces, P and P_1. What shall be the magnitude of these two forces to keep the ball suspended in just this position? We have here just the opposite of our last

case; and, instead of finding the diagonal of the resultant, we have the diagonal, which is the line W, and wish to find the sides of the parallelogram. To do this, prolong P and P_1, and from B draw lines parallel to them to complete the parallelogram. Then will CA be the required magnitude for P, and CB for P_1.

Thus we see how one force can be made to have the same effect as many, or many can be made to do the work of one. Bearing the above in mind, we are now prepared to study the following propositions:

I. *A force may be represented by a straight line.*

In considering the action of forces, either in relation to structures or by themselves, it is very convenient to represent the force graphically, which can easily be done by a straight line having an arrow-head, as in Fig. 3. The length of the line, if drawn to a scale of pounds, shows the value of the force in pounds; the direction of the line indicates the direction of the force; the arrow-head shows which way it acts; and the point A denotes the point of application. Thus we have the direction, magnitude, and point of application of the force represented, which is all that we need to know.

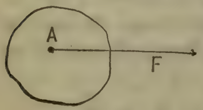


Fig. 3

Parallelogram of Forces.—II. *If two forces applied at one point, and acting in the same plane, be represented by two straight lines inclined to each other, their resultant will be equal to the diagonal of the parallelogram formed on these lines.*

Thus, if the lines AB and AC (Fig. 4) represent two forces acting on one point, A, and in the same plane, then, to obtain the force which would have the same effect as the two forces, we complete the parallelogram $ABDC$, and draw the diagonal AD. This line will then represent the resultant of the two forces.

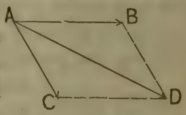


Fig. 4

When the two given forces are at right angles to each other, the resultant will, by geometry, be equal to the square root of the sum of the squares of the other two forces.

The Triangle of Forces.—III. *If three forces acting on a point be represented in magnitude and direction by the sides of a triangle taken in order, they will keep the point in equilibrium.*

Thus let P, Q, and R (Fig. 5) represent three forces acting on the point O. Now, if we can draw a triangle like that shown at

the right of Fig. 5, whose sides shall be respectively parallel to the forces, and shall have the same relation to each other as do the forces, then the forces will keep the point in equilibrium. If such a triangle cannot be drawn, the forces will be unbalanced, and the point will not be in equilibrium.

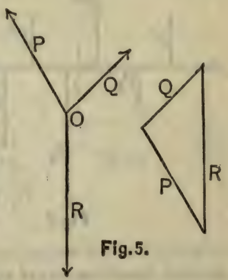


Fig. 5.

The Polygon of Forces.—IV. *If any number of forces acting at a point can be represented in magnitude and direction by the sides of a polygon taken in order, they will be in equilibrium.*

This proposition is only the preceding one carried to a greater extent.

Moments.—In considering the stability of structures and the strength of materials, we are often obliged to take into consideration the *moments* of the forces acting on the structure or piece; and a knowledge of what a moment is, and the properties of moments, is essential to the proper understanding of these subjects.

When we speak of the moment of a force, we must have in mind some fixed point about which the moment is taken.

The moment of a force about any given point may be defined as the product of the force into the perpendicular distance from the point to the line of action of the force; or, in other words, the moment of a force is the *product of the force by the arm with which it acts.*

Thus if we have a force F (Fig. 6), and wish to determine its moment about a point P, we determine the perpendicular distance Pa, between the point and the line of action of the force, and multiply it by the force in pounds. For example, if the force F were equal to a weight of 500 pounds, and the distance Pa were 2 inches, then the moment of the force about the point P would be 1,000 inch-pounds.

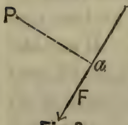


Fig. 6

The following important propositions relating to forces and moments should be borne in mind in calculating the strength or stability of structures.

V.—*If any number of parallel forces act on a body, that the body shall be in equilibrium, the sum of the forces acting in one direction must equal the sum of the forces acting in the opposite direction.*

Thus if we have the parallel forces P^1, P^2, P^3, and P^4, acting on the rod AB (Fig. 7), in the opposite direction to the forces P_1, P_2, P_3, then, if the rod is in equilibrium, the sum of the forces P^1, P^2, P^3, and P^4, must equal the sum of the forces P_1, P_2, and P_3.

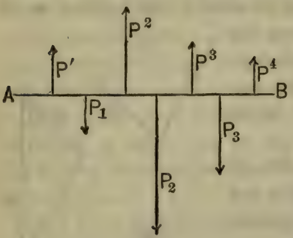


Fig.7

VI. *If any number of parallel forces act on a body in opposite directions, then, for the body to be in equilibrium, the sum of the moments tending to turn the body in one direction must equal the sum of the moments tending to turn the body in the opposite direction about any given point.*

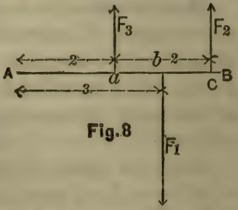


Fig.8

Thus let Fig. 8 represent three parallel forces acting on a rod AB. Then for the rod to be in equilibrium, the sum of the forces F_2 and F_3 must be equal to F_1. Also, if we take the end of the rod, A, for our axis, then must the moment of F_1 be equal to the sum of the moments of F_2 and F_3 about that point, because the moment of F_1 tends to turn the rod down to the right, and the moments of F_2 and F_3 tend to turn the rod up to the left, and there should be no more tendency to turn the rod one way than the other. For example, let the forces F_3, F_2, each be represented by 5, and let the distance Aa be represented by 2, and the distance Ac by 4. The force F_1 must equal the sum of the forces F_3 and F_2, or 10; and its moment must equal the sum of the moments of F_3 and F_2. If we take the moments around A, then the moment of $F_3 = 5 \times 2 = 10$, and of $F_2 = 5 \times 4 = 20$. Their sum equals 30: hence the moment of F_1 must be 30. Dividing the moment 30 by the force 10, we have for the arm 3; or the force F_1 must act at a distance 3 from A to keep the rod in equilibrium.

If we took our moments around b, then the force F_1 would have no moment, not having any arm, and so the moment of F_2 about b must equal the moment of F_3 about the same point; or, as in this case the forces are equal, they must both be applied at the same distance from b, showing that b must be halfway between a and c, as was proved before.

The Principle of the Lever.—This principle is based upon the two preceding propositions, and is of great importance and convenience.

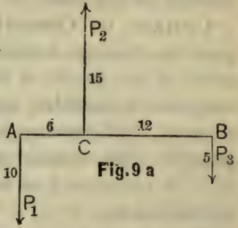


Fig. 9 a

VII. *If three parallel forces acting in one plane balance each other, then each force must be proportional to the distance between the other two.*

Thus, if we have a rod AB (Figs. 9a, 9b, and 9c), with three forces, P_1, P_2, P_3, acting on it, that the rod shall be balanced, we must have the following relation between the forces and their points of application; viz.,

$$\frac{P_1}{CB} : \frac{P_2}{AB} : \frac{P_3}{AC};$$

or

$$P_1 : P_2 : P_3 :: BC : AB : AC.$$

This is the case of the common lever, and gives the means of determining how much a given lever will raise.

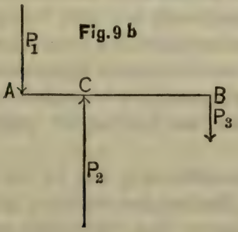


Fig. 9 b

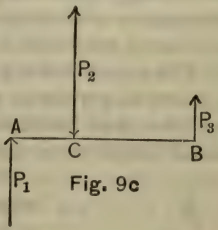


Fig. 9c

The proportion is also true for any arrangement of the forces (as shown in Figs. a, b, and c), provided, of course, the forces are lettered in the order shown in the figures.

Example.—Let the distance AC be 6 inches, and the distance CB be 12 inches. If a weight of 500 pounds is applied at the point B, how much will it raise at the other end, and what support will be required at C (Fig. 9b)?

Ans. Applying the rule just given, we have the proportion:—

$$P_3 : P_1 :: AC : CB, \text{ or } 500 : (P_1) :: 6 : 12.$$

Hence $P_1 = 1{,}000$ pounds; or 500 pounds applied at B will lift 1,000 suspended at A. The supporting force at C must, by proposi-

tion V., be equal to the sum of the forces P_1 and P_3, or 1,500 pounds in this case.

Centre of Gravity.—The lines of action of the force of gravity converge towards the centre of the earth; but the distance of the centre of the earth from the bodies which we have occasion to consider, compared with the size of those bodies, is so great, that we may consider the lines of action of the forces as parallel. The number of the forces of gravity acting upon a body may be considered as equal to the number of particles composing the body.

The centre of gravity of a body may be defined as the point through which the resultant of the parallel forces of gravity, acting upon the body, passes in every position of the body.

If a body be supported at its centre of gravity, and be turned about that point, it will remain in equilibrium in all positions. The resultant of the parallel forces of gravity acting upon a body is obviously equal to the weight of the body, and if an equal force be applied, acting in a line passing through the centre of gravity of the body, the body will be in equilibrium.

Examples of Centres of Gravity.—*Centre of Gravity of Lines. Straight Lines.*—By a line is here meant a material line whose transverse section is very small, such as a very fine wire.

The centre of gravity of a uniform straight line is at its middle point. This proposition is too evident to require demonstration.

The centre of gravity of the perimeter of a triangle is at the centre of the circle inscribed in the lines joining the centres of the sides of the given triangle.

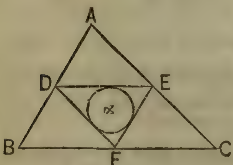


Fig. 10

Thus, let ABC (Fig. 10) be the given triangle. To find the centre of gravity of its perimeter, find the middle points, D, E, and F, and connect them by straight lines. The centre of the circle inscribed in the triangle formed by these lines will be the centre of gravity sought.

Symmetrical Lines.—The centre of gravity of lines which are symmetrical with reference to a point will be at that point. Thus the centre of gravity of the circumference of a circle or an ellipse is at the geometrical centre of those figures.

The centre of gravity of the perimeter of an equilateral triangle, or of a regular polygon, is at the centre of the inscribed circle.

The centre of gravity of the perimeter of a square, rectangle, or parallelogram, is at the intersection of the diagonals of those figures.

Centre of Gravity of Surfaces. Definition.—A surface here means a very thin plate or shell.

Symmetrical Surfaces.—If a surface can be divided into two symmetrical halves by a line, the centre of gravity will be on that line: if it can be divided by two lines, the centre of gravity will be at their intersection.

The centre of gravity of the surface of a circle or an ellipse is at the geometrical centre of the figure, of an equilateral triangle or a regular polygon, it is at the centre of the inscribed circle; of a parallelogram, at the intersection of the diagonals; of the surface of a sphere, or an ellipsoid of revolution, at the geometrical centre of the body; of the convex surface of a right cylinder at the middle point of the axis of the cylinder.

Irregular Figures.—Any figure may be divided into rectangles and triangles, and, the centre of gravity of each being found, the centre of gravity of the whole may be determined by treating the centres of gravity of the separate parts as particles whose weights are proportional to the areas of the parts they represent.

Triangle.—To find the centre of gravity of a triangle, draw a line from each of two angles to the middle of the side opposite: the intersection of the two lines will give the centre of gravity.

Quadrilateral.—To find the centre of gravity of any quadrilateral, draw diagonals, and, from the end of each farthest from their intersection, lay off, toward the intersection, its shorter segment: the two points thus formed with the point of intersection will form a triangle whose centre of gravity is that of the quadrilateral.

Thus, let Fig. 11 be a quadrilateral whose centre of gravity is sought. Draw the diagonals AD and BC, and from A lay off $AF = ED$, and from B lay off $BH = EC$. From E draw a line to the middle of FH, and from F a line to the middle of EH. The point of intersection of these two lines is the centre of gravity of the quadrilateral. This is a method commonly used for finding the centre of gravity of the voussoirs of an arch.

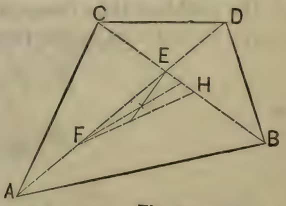


Fig. 11

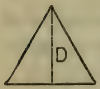
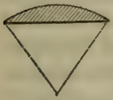


Table of Centres of Gravity.—Let a denote a line drawn from the vertex of a figure to the middle point of the base, and D the distance from the vertex to the centre of gravity. Then



Segment.

Sector.

In an isosceles triangle	$D=\frac{2}{3}a$
In a segment of a circle, Vertex at centre of circle . . .	$D=\dfrac{\text{chord}^3}{12\times\text{area}}$
In a sector of a circle, the vertex being at the centre	$D=R\times\dfrac{2\times\text{chord}}{3\times\text{arc}}$
In a semicircle, vertex being at the centre	$D=\dfrac{4R}{3\pi}=0.4244R$
In a quadrant of a circle . . .	$D=\frac{3}{5}R$
In a semi-ellipse, vertex being at the centre .	$D=0.425a$
In a parabola, vertex at intersection of axis with curve .	$D=\frac{3}{5}a$
In a cone or pyramid	$D=\frac{3}{4}a$

In a frustum of a cone or pyramid, let h=height of complete cone or pyramid, h_1=height of frustum, and the vertex be at apex of complete cone or pyramid; then $D=\dfrac{3(h^4-h_1^4)}{4(h^3-h_1^3)}$.

Centre of Gravity of Heavy Particles.—*Centre of Gravity of Two Particles.*—Let P be the weight of a particle at A (Fig. 12), and W that at C.

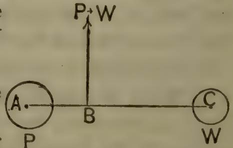


Fig. 12

The centre of gravity will be at some point, B, on the line joining A and C. The point B must be so situated, that if the two particles were held together by a stiff wire, and were supported at B by a force equal to the sum of P and W, the two particles would be in equilibrium.

The problem then comes under the principle of the lever, and hence we must have the proportion (see proposition VII.).

$$P+W : P :: AC : BC,$$

or

$$BC=\frac{P\times AC}{P+W}.$$

If $W=P$, then $BC=AB$, or the centre of gravity will be halfway between the two particles. This problem is of great importance, for it presents itself in many practical examples.

Centre of Gravity of Several Heavy Particles.—Let W_1, W_2, W_3, W_4 and W_5 (Fig. 13) be the weights of the particles.

Join W_1 and W_2 by a straight line, and find their centre of gravity A, as in the preceding problem. Join A with W_3, and find the centre of gravity B, which will be the centre of gravity of the three weights W_1, W_2, and W_3. Proceed in the same way with each weight, and the last centre of gravity found will be the centre of gravity of all the particles.

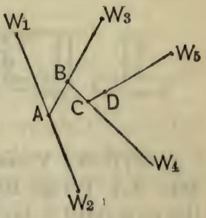


Fig. 13

In both of these cases the lines joining the particles are supposed to be horizontal lines, or else the horizontal projection of the real straight line which would join the points.

Centre of Gravity of Compound Sections Found by Moments.—To determine the strength of a beam having an unsymmetrical section, it is first necessary to determine the distance of the centre of gravity of the section from the bottom of the beam. Various other computations also involve finding the centre of gravity of an irregular figure, so that the problem is one of practical importance.

If the figure of which the centre of gravity is sought can be divided into regular figures the readiest and simplest method of finding the distance of the centre of gravity from one edge is by means of moments.

To explain this method we will assume a T-shape section of uniform thickness pivoted on a wire, XX, as in Fig. 14. The T

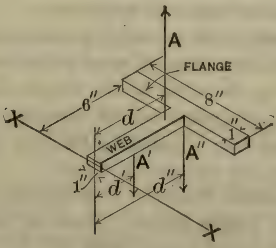


Fig. 14

is made up of two rectangles, one forming what we will call the flange, the other the web. The centre of gravity of each of

these rectangles will be at their centre, which can easily be found.

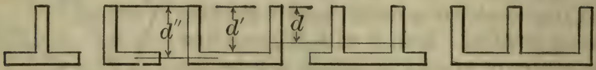


Fig. 15

Now, if the T were placed horizontally, as in the figure, the axis XX being fixed, it would immediately revolve about the axis until it became vertical, and the moments causing the revolution would be $A'd'+A''d''$, A' representing the weight of the web and A'' the weight of the flange. To hold the T in a horizontal position, there must be a moment acting in the opposite direction just equal to the sum of the two moments acting downwards, and if the force in this upward moment, represented by A, is equal to the weight of the entire T, then the force must be applied *at the centre of gravity* of the entire figure to make its moment just equal to the sum of the two moments.

But the moment of A will be Ad, therefore d must denote the distance from the end of the web to the centre of gravity of the entire figure, and as $Ad=A'd'+A''d''$, d must equal

$$\frac{A'd'+A''d''}{A}, \tag{1}$$

and A is equal to $A'+A''$. As the weight of any homogeneous material of uniform thickness is proportional to the area; A, A', and A'' may be used to represent areas as well as weights.

To reduce formula 1 to a rule we have:

VIII. *The distance of the centre of gravity of a compound figure from any line, taken as a base, is equal to the sum of the products, found by multiplying the areas of the simple figures of which the compound figure is composed by the distance of their centres of gravity from the base line, divided by the area of the entire figure.* This rule applies to any compound figure.

Example I.—Assume that the T shown in Fig. 14 has the dimensions indicated in the figure. Then A' will equal 6, A'', 8, and A, 14. d' will equal 3 and d'' $6\frac{1}{2}$.

The sum of the products of A' by d' and A'' by d' will be $18+52$ or 70, and this divided by 14, the area of the entire figure, gives 5 ins. for the distance d.

The distance d of the centre of gravity from the top of the

webs, in any of the figures shown in Fig. 15, may be found by the following formula:

$$d=\frac{\text{area of webs}\times\frac{d'}{2}+\text{area of flange }\times d''}{\text{area of webs }+\text{area of flange}}. \qquad (2)$$

For a section like that shown in Fig. 16, in which A', A'', A''' represent the area of the respective rectangles, the distance d of the centre of gravity from the top may be found by the formula

$$d=\frac{A'\times d'+A''\times d''+A'''\times d'''}{A'+A''+A'''} \qquad (3)$$

Example II.—To show the application of proposition VIII. to any compound figure, we will take that shown by Fig. 17 and find the distance d of the centre gravity of the entire figure from the vertex o. The area of the triangle is 36 sq. ins. and of the semicircle 56.5.

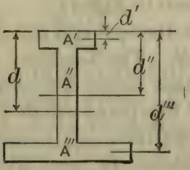


Fig. 16

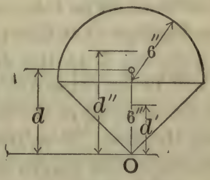


Fig. 17

From the table on page 238 we find that the distance of the centre of gravity of an isosceles triangle from the vertex is $\frac{2}{3}$ its height, which gives 4 as the value for d'. The centre of gravity for a semicircle is $0.4244\ r$ from its base, so that d'' equals 8.54. Then

$$d=\frac{36\times4+56.5\times8.54}{36+56.5}=6.77.$$

This method of finding the centre of gravity is the same as that given in Chapter IX. for finding supporting forces, except that in the latter case the problem is to find the balancing force instead of the arm.

CHAPTER VII.

STABILITY OF PIERS AND BUTTRESSES.

A PIER or buttress may be considered stable when the forces acting upon it do not cause it to rotate or "tip over," or any course of stones or brick to slide on its bed. When a pier has to sustain only a vertical load, it is evident that the pier must be stable, although it may not have sufficient strength.

It is only when the pier receives a thrust, such as that from a rafter or an arch, that its stability must be considered.

In order to resist rotation, we must have the condition that the moment of the thrust of the pier about any point in the outside of the pier shall not exceed the moment of the weight of the pier about the same point.

To illustrate let us take the pier shown in Fig. 1.

Let us suppose that this pier receives the foot of a rafter which exerts a thrust T in the direction AB. The tendency of this thrust will be to cause the pier to rotate about the outer edge b_1, and the moment of the thrust about this point will be $T \times a'b_1$, $a'b_1$ being the arm. Now, that the pier shall be just in equilibrium, the moment of the weight of the pier about the same edge must just equal $T \times a'b_1$. The weight of the pier will, of course, act through the centre of gravity of the pier (which in this case is at the centre), and in a vertical direction; and its arm will be b_1c, or one-half the thickness of the pier.

Hence, to have equilibrium, we must have the equation

$$T \times a^1b_1 = W \times b_1c.$$

But under this condition the least additional thrust, or the crushing off of the outer edge, would cause the pier to rotate; hence, to have the pier in safe equilibrium, we must use some factor of safety.

This is generally done by making the moment of the weight equal to that of the thrust when referred to a point in the bottom of the pier, a certain distance in from the outer edge.

This distance for piers or buttresses should not be less than one-fourth of the thickness of the pier.

Representing this point in the figure by b, we have the necessary equation for the safe stability of the pier,

$$T \times ab = W \times \tfrac{1}{4}t,$$

t denoting the width of the pier.

We cannot from this equation determine the dimensions of a pier to resist a given thrust, because we have the distance ab, t, and W, all unknown quantities. Hence we must first guess at the size of the pier, then find the length of the line ab, and see if the moment of the pier is equal to that of the thrust. If it is not we must guess again.

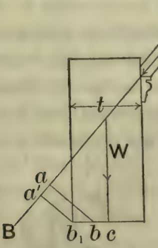


Fig. 1.

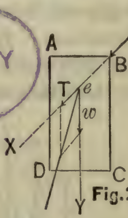


Fig. 2

Graphic Method of Determining the Stability of a Pier or Buttress.—When it is desired to determine if a given pier or buttress is capable of resisting a given thrust, the problem can easily be solved graphically in the following manner.

Let $ABCD$ (Fig. 2) represent a pier which sustains a given thrust T at B.

To determine whether the pier will safely sustain this thrust, we proceed as follows.

Draw the indefinite line BX in the direction of the thrust. Through the centre of gravity of the pier (which in this case is at the centre of the pier) draw a vertical line until it intersects the line of the thrust at e. As a force may be considered to act anywhere in its line of direction, we may consider the thrust and the weight to act at the point e, and the resultant of these two forces can be obtained by laying off the thrust T from e on eX, and the weight of the pier W, from e on the line eY, both to the same scale (pounds to the inch), completing the parallelogram, and drawing the diagonal. If this diagonal prolonged cuts the base of the pier at less than one-fourth of the width of the base from the outer edge, the pier will be unstable and its dimensions must be changed.

The stability of a pier may be increased by adding to its weight (by placing some heavy material on top) or by increasing its width at the base by means of "off-sets," as in Fig. 3A.

Figs. 3 (A and B) show the method of determining the stability of a buttress with offsets.

The first step is to find the vertical line passing through the centre of gravity of the whole pier. This is best done by dividing the buttress up into quadrilaterals, as *ABCD, DEFG,* and *GHIK* (Fig. 3A), finding the centre of gravity of each quadrilateral by the method of diagonals explained in Chapter VI. and then measuring the perpendicular distances X_1, X_2, X_3 from the different centres of gravity to the line *KI*.

Multiply the area of each quadrilateral by the distance of its centre of gravity from the line *KI* and add together the areas and the products. Divide the sum of the latter by the sum of the former and the result will be the distance of the centre of gravity of the whole buttress from *KI*. This distance we denote by X_0.

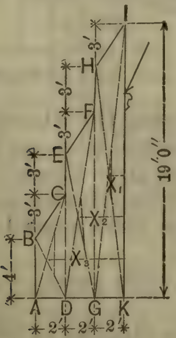


Fig. 3 A

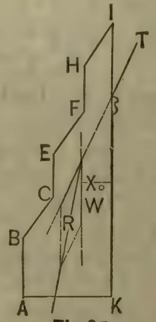


Fig. 3 B

EXAMPLE I.—Let the buttress shown in Fig. 3A have the dimensions given between the cross-marks. Then the area of the quadrilaterals and the distances from their centres of gravity to *KI* would be as follows:

1st area = 35 sq. ft.	$X_1 = 0'.95$	1st area $\times X_1 = 33.25$
2d area = 23 sq. ft.	$X_2 = 2'.95$	2d area $\times X_2 = 67.85$
3d area = 11 sq. ft.	$X_3 = 4'.95$	3d area $\times X_3 = 54.45$
Total area, 69 sq. ft.		Total moments, 155.55

The sum of the moments is 155.55, and, dividing this by the total area, we have 2.25 as the distance X_0. Measuring this to

the scale of the drawing from KI, we have a point through which the vertical line passing through the centre of gravity must pass. After this line is found, the method of determining the stability of the pier is the same as that given for the pier in Fig. 2. Fig. 3B also illustrates the method. If the buttress is more than one foot thick (at right angles to the plane of the paper), the cubic contents of the buttress must be obtained to find the weight. It is easier, however, to divide the real thrust by the thickness of the buttress, which gives the thrust per foot of buttress.

Line of Resistance.—*Definition.* The line of resistance or of pressures of a pier or buttress is a line drawn through the centre of pressure of each joint.

The *centre of pressure* of any joint is the point where the resultant of the forces acting on that portion of the pier above the joint cuts it.

The line of pressures, or of resistance, when drawn in a pier, shows how near the greatest stress on any joint comes to the edges of that joint.

It can be drawn by the following method:

Let $ABCD$ (Fig. 4) be a pier whose line of resistance we wish to draw. First divide the pier in height, into portions two or three feet high, by drawing horizontal lines. It is more convenient to make the portions all of the same size.

Prolong the line of the thrust, and draw a vertical line through the centre of gravity of the pier, intersecting the line of thrust at the point a. From a lay off to a scale the thrust T and the weights of the different portions of the pier, commencing with the weight of the upper portion. Thus w_1 represents the weight of the portion above the first joint; w_2 represents the weight of the second portion; and so on. The sum of the w's will equal the whole weight of the pier.

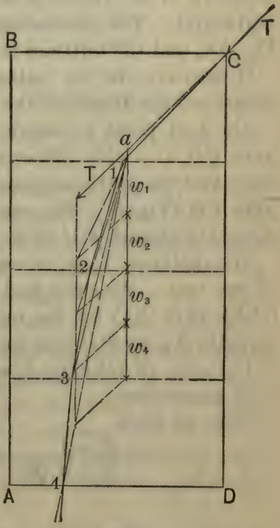


Fig. 4

Having proceeded thus far, complete a parallelogram, with T

and w_1 for its two sides. Draw the diagonal, and prolong it. Where it cuts the first joint will be a point in the line of resistance. Draw another parallelogram, with T and w_1+w_2 for its two sides. Draw the diagonal intersecting the second joint at 2. Proceed in this way, when the last diagonal will intersect the base in 4. Join the points 1, 2, 3, and 4, and the resulting line will be the line of resistance.

We have taken the simplest case as an example; but the same principle is true for any case.

Should the line of resistance of a pier at any point approach the outside edge of the joint nearer than one-quarter the width of the joint, the pier should be considered unsafe.

As an example embracing all the principles given above we will take the following case.

EXAMPLE II.—Let Fig. 5 represent the section of a side wall of a church, with a buttress against it. Opposite the buttress, on the inside of the wall, is a hammer-beam truss, which we will suppose exerts an outward thrust on the walls of the church amounting to about 9600 pounds. We will further consider that the resultant of the thrust acts at P, and at an angle of 60° with a horizontal. The dimensions of the wall and buttress are given in Fig. 5A, and the buttress is two feet thick.

QUESTION.—Is the buttress sufficient to enable the wall to withstand the thrust of the truss?

The first point to decide is if the line of resistance cuts the joint CD at a safe distance in from C. To ascertain this we must find the centre of gravity of the wall and buttress above the joint CD (Fig. 5). We can find this easiest by the method of moments around KM (Fig. 5A), as already explained.

The distance X_1 is, of course, half the thickness of the wall. or one foot. We next find the centre of gravity of the portion $CEFG$ (Fig. 5A) by the method of diagonals, and, scaling the distance X_2, we find it to be 2.95 feet.

The area of $CEFG=A_2=10$ square feet; and of $GIKL=A_1=26$ square feet.

Then we have

$$\begin{array}{lll} X_1=1 & A_1=26 & A_1\times X_1=26 \\ X_2=2.95 & A_2=10 & A_2\times X_2=29.5 \\ & \overline{36} & 36)\overline{55.5} \\ & & \overline{X_0=1.5} \end{array}$$

Or the centre of gravity is at a distance 1.5 feet from the line *ED* (Fig. 5). Then on Fig. 5 measure the distance $X_0 = 1.5$ feet, and through the point *a* draw a vertical line intersecting the line of the thrust prolonged at *O*. Now, if the thrust is 9600 pounds for a buttress two feet thick, it would be half that, or 4800 pounds, for a buttress one foot thick. We will call the weight of the masonry of which the buttress and wall is built 150 pounds per cubic foot. Then the thrust is equivalent to $4800 \div 150$, or 32 cubic feet of masonry. Laying this off to a scale from *O*, in the direction of the thrust and the area of the masonry, 36 square feet from *O* on the vertical line, completing the rectangle, and drawing the diagonal, we find it cuts the joint *CD* at *t*, within the limits of safety.

We must next find where the line of resistance cuts the base *AB*.

First find the centre of gravity of the whole figure, which is found by ascertaining the distances X_2', X_3', in Fig. 5A, and making the following computation.

$$\begin{array}{lll}
X_1' = 1' & A_1' = 40 & A_1' \times X_1' = 40 \\
X_2' = 2'.98 & A_2' = 24 & A_2' \times X_2' = 71.52 \\
X_3' = 4'.95 & A_3' = 12 & A_3' \times X_3' = 59.40 \\
\hline
 & \quad 76 & 76)170.92 \\
\hline
 & & X_0' = 2.25
\end{array}$$

Then from the line *EB* (Fig. 5) lay off the distance $X_0' = 2'.25$, and draw through *d* a vertical line intersecting the line of the thrust at O'. On this vertical from O' measure down the whole area 76, and from its extremity lay off the thrust $T = 32$ at the proper angle. Draw the line $O'e$ intersecting the base at *c*. This is the point where the line of resistance cuts the base; and, as it is at a safe distance in from *A*, the buttress has sufficient stability.

If there were more offsets, we should proceed in the same way, finding where the line of resistance cuts the joint at the top of each offset. The reason for doing this is because the line of resistance might cut the base at a safe distance from the outer edge, while higher up it might come outside of the buttress, so that the buttress would be unstable.

The method given in these examples is applicable to piers of any shape or material.

Should the line of resistance make an angle less than 30° with

any joint, it might cause the stones above the joint to slide on their bed. This can be prevented either by dowelling, or by inclining the joint.

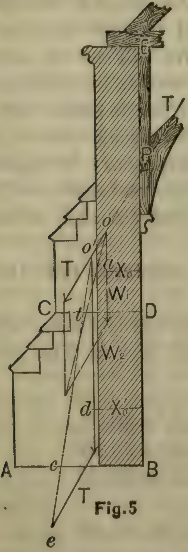


Fig.5

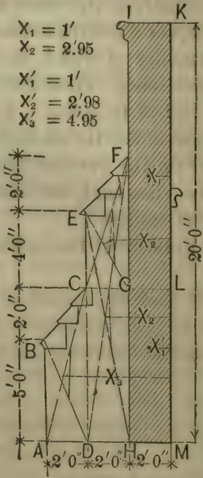


Fig.5 A

It is very seldom in architectural construction that such a case would occur, however.

CHAPTER VIII.

THE STABILITY OF ARCHES.

The arch is an arrangement for spanning large openings by means of small blocks of stone, or other material, arranged in a particular way. As a rule, the arch answers the same purpose as the beam, but it is widely different in its action and in the effect that it has upon the appearance of an edifice. A beam exerts merely a vertical force upon its supports, but the arch exerts both a vertical load and an outward thrust. It is this thrust which requires that the arch should be used with caution where the abutments are not abundantly large.

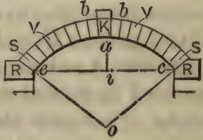


Fig. 1

Before taking up the principles of the arch, we will define the many terms relating to it. The distance *ec* (Fig. 1) is called the *span* of the arch; *ai*, its rise; *b*, its *crown;* its lower boundary line, *eac*, its *soffit* or *intrados;* the outer boundary line, its *back* or *extrados. The terms "soffit"* and "back" are also applied to the entire lower and upper curved *surfaces* of the whole arch. The ends of the arch, or the sides which are seen, are called its *faces.* The blocks of which the arch itself is composed are called *voussoirs:* the centre one, *K*, is called the *keystone;* and the lowest ones, *SS*, the *springers.* In *segmental* arches, or those whose intrados is not a complete semicircle, the springers generally rest upon two stones, as RR, which have their upper surface cut to receive them: these stones are called *skewbacks.* The line connecting the lower edges of the springers is called the *springing-line;* the sides of the arch are called the *haunches;* and the load in the triangular space, between the haunches and a horizontal line drawn from the crown, is called the *spandrel.*

The blocks of masonry, or other material, which support two successive arches, are called *piers:* the extreme blocks, which, in the case of stone bridges, generally support on one side embankments of earth, are called *abutments.*

A pier strong enough to withstand the thrust of either arch, should the other fall down, is sometimes called an *abutment pier*

Besides their own weight, arches usually support a permanent load or surcharge of masonry or of earth.

In using arches in architectural constructions, the form of the arch is generally governed by the style of the edifice, or by a limited amount of space. The semicircular and segmental forms of arches are the best as regards stabilty, and are the simplest to construct. Elliptical and three-centred arches are not as strong as circular arches, and should only be used where they can be given all the strength desirable.

The strength of an arch depends very much upon the care with which it is built and of the quality of the work.

In stone arches, special care should be taken to cut and lay the beds of stones accurately, and to make the bed-joints thin and close, in order that the arch may be strained as little as possible in settling.

To insure this, arches are sometimes built dry, *grout* or liquid mortar being afterwards run into the joints; but the advantage of this method is doubtful.

Brick Arches may be built either of wedge-shaped bricks, moulded or rubbed so as to fit to the radius of the soffit, or of bricks of common shape. The former method is undoubtedly the best, as it enables the bricks to be thoroughly bonded, as in a wall; but, as it involves considerable expense to make the bricks of the proper shape, this method is very seldom employed. Where bricks of the ordinary shape are used, they are accommodated to the curved figure of the arch by making the bed-joints thinner towards the intrados than towards the extrados; or, if the curvature is sharp, by driving thin pieces of slate into the outer edges of those joints; and different methods are followed for bonding them. The most common way is to build the arch in concentric rings, each half a brick thick; that is, to lay the bricks all stretchers, and to depend upon the tenacity of the mortar or cement for the connection of the several rings. This method is deficient in strength, unless the bricks are laid in cement at least as tenacious as themselves. Another way is to introduce courses of headers at intervals, so as to connect pairs of half-brick rings together.

This may be done either by thickening the joints of the outer of a pair of half-brick rings with pieces of slate, so that there shall be the same number of courses of stretchers in each ring between two courses of headers, or by placing the courses of headers at such distances apart, that between each pair of them there shall

be one course of stretchers more in the outer than in the inner ring.

The former method is best suited to arches of long radius; the latter, to those of short radius. *Hoop iron* laid round the arch, between half-brick rings, as well as longitudinally and radially, is very useful for strengthening brick arches. The bands of hoop iron which traverse the arch radially may also be bent, and prolonged in the bed-joints of the backing and spandrels.

By the aid of hoop-iron bond, Sir Marc-Isambard Brunel built a half-arch of bricks laid in strong cement, which stood, projecting from its abutment like a bracket, to the distance of sixty feet, until it was destroyed by its foundation being undermined.

The New-York City Building Laws make the following requirements regarding brick arches:—

"All arches shall be at least four inches thick. Arches over four-foot span shall be increased in thickness toward the haunches by additions of four inches in thickness of brick. The first additional thickness shall commence at two and a half feet from the centre of the span, the second addition at six and a half feet from the centre of the span; and the thickness shall be increased thence four inches for every additional four feet of span towards the haunches.

"The said brick arches shall be laid to a line on the centres with a close joint, and the bricks shall be well wet, and the joints filled with cement mortar in proportions of not more than two of sand to one of cement by measure. The arches shall be well grouted and pinned, or chinked with slate, and keyed." *

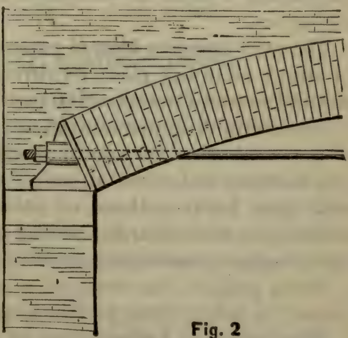


Fig. 2

Rule for Radius of Brick Arches.—A good rule for the radius of segmental brick arches over windows, doors, and other small openings is to *make the radius equal to the width of the opening.* This gives a good rise to the arch and makes a pleasing proportion to the eye.

* For illustrations of the different ways of building brick arches, see Chapter VII. of Part I., "Building Construction and Superintendence."

Segmental Arches with Tie Rods.—It is often desirable to span openings in a wall by means of an arch when there is not sufficient abutments to withstand the thrust or kick of the arch. In such a case the arch can be formed on two cast-iron skewbacks, which are held in place by iron rods, as is shown in Fig. 2.

When this is done, it is necessary to proportion the size of the rods to the thrust of the arch. The horizontal thrust of the arch is very nearly represented by the following formula:

$$\text{Horizontal thrust} = \frac{\text{load on arch} \times \text{span}}{8 \times \text{rise of arch in feet}}.$$

If the load is concentrated at the centre of the arch, the thrust will be twice that given by above formula.

The stress in the rod or rods will equal the horizontal thrust of the arch; if there are two rods, the stress in each will be one-half the thrust; if there are three rods, then each must be capable of resisting $\frac{1}{3}$ of the thrust. Knowing the stress in the rods, their size may be readily determined from Table III. of Chapter XI.

Centres for Arches.—*A centre* is a temporary structure, generally of timber, by which the voussoirs of an arch are supported while the arch is being built. It consists of parallel frames or ribs, placed at convenient distances apart, curved on the outside to a line parallel to that of the soffit of the arch, and supporting a series of transverse blanks, upon which the arch stones rest.

The most common kind of centre is one which can be lowered, or struck all in one piece, by driving out wedges from below it, so as to remove the support from every point of the arch at once.

The centre of an arch should not be struck until the solid part of the backing has been built and the mortar has had time to set and harden; and, when an arch forms one of a series of arches with piers between them, no centre should be struck so as to leave a pier with an arch abutting against one side of it only, unless the pier has sufficient stability to act as an abutment.

When possible, the centre of a large brick arch should not be struck for two or three months after the arch is built.

Mechanical Principles of the Arch.—In designing an arch, the first question to be settled is the form of the arch; and in regard to this there is generally but little choice. Where the abutments are abundantly large, the segmental arch is the strongest form; but where it is desired to make the abutments of the

arch as light as possible, a pointed or semicircular arch should be used.

Depth of Keystone.—Having decided upon the form of the arch, the depth of the arch-ring must next be decided. This is generally determined by computing the required depth of keystone, and making the whole ring of the same or a little larger depth.

In considering the strength of an arch, the depth of the keystone is considered to be only the distance from the extrados to the intrados of the arch; and if the keystone projects above the arch-ring, as in Fig. 1, the projection is considered as a part of the load on the arch.

There are several rules for determining the depth of the keystone, but all are empirical; and they differ so greatly that it is difficult to recommend any particular one. Professor Rankine's rule is often quoted, and is probably true enough for most arches. It applies to both circular and elliptical arches, and is as follows:

Rankine's Rule.—*For the depth of the keystone,* take a mean proportional between the inside radius at the crown, and 0.12 of a foot for a single arch, and 0.17 of a foot for an arch forming one of a series. Or, if represented by a formula,

Depth of keytsone for a single arch in feet

$$=\sqrt{(0.12 \times \text{radius at crown})}.$$

Depth of keystone for an arch of a series in feet

$$=\sqrt{(0.17 \times \text{radius at crown})}.$$

This rule seems to agree very well with actual cases in arches of a certain kind. By it, however, the depth of keystone is the same for spans of any length, provided the radius is the same; and in this particular, it seems to us, the rule is not satisfactory.

Trautwine's Rule.—Mr. Trautwine, from calculations made on a large number of arches, deduced an original rule for the depth of keystone, which is more agreeable to theory than Rankine's. His rule is, for *cut stone,*

$$\text{Depth of key in feet} = \left(\frac{\sqrt{\text{radius} + \text{half span}}}{4}\right) + 0.2 \text{ foot}.$$

For *second-class* work this depth may be increased about one-eighth part, or for *brick or fair rubble,* about one-fourth.

The following table gives a few examples of the depth of keystone of some existing bridges, together with the depth which

would be required by Trautwine's or Rankine's Rule. From this table it will be seen that both rules agree very well with practice.

TABLE I.—SHOWING DEPTH OF KEYSTONE OF SOME EXISTING ARCHES.

Bridge (circular arc).	Span.	Rise.	Radius.	Actual depth of key.	Calculated depth of key. Trautwine's Rule.	Rankine's Rule.	Engineer.
	Ft.	Ft.	Ft.	Ft.	Ft.	Ft.	
Cabin John, Washington Aqueduct......	220.0	57.25	134.25	4.60	4.11	4.00	Meigs.
Grosvenor Bridge, Chester, Eng......	200.0	42.00	140.00	4.00	4.07	4.10	Hartley.
Dora Riparia, Turin, Italy............	148.0	18.00	160.10	4.92	4.03	4.38	Mosca.
Tongueland, England.	118 0	38.00	64.80	3.50	3.00	2.79	Telford.
Dean Bridge, Scotl'nd, in a series.........	90.0	30.00	48.90	3.00	2.62	2.88	Telford.
Falls Bridge, Philadelphia & Reading Railroad...........	78.0	25.00	43.00	3.00	2.46	2.27	Steele.
Chestnut St. Bridge, Philadelphia, brick in cement........	60.0	18 00	34.00	2.50	2.20	2.00	Kneass.
Philadelphia & Reading Railroad......	44.0	8.00	34.30	2.50	2.08	2.02	Steele.
Philadelphia & Reading Railroad......	31.2	5.00	26.80	1.66	1.83	1.79	Steele.

Table II., taken from Trautwine's "Civil Engineers' Handbook," gives the depth of keystone for arches of first-class cut-stone, according to Trautwine's Rule. For second-class cut-stone add about one-eighth part and for good rubble or brick, about one-fourth part.

Having decided what the thickness of the arch-ring will be it remains to determine whether such an arch would be stable if built.

The following example will illustrate the method of determining this point:

EXAMPLE I.—*Unloaded semicircular arch of 20-foot span.*

First, to find the depth of keystone, we will take Rankine's Rule, and by it we have

$$\text{Depth of key} = \sqrt{0.12 \times 10} = \sqrt{1.2} = 1.1 \text{ feet.}$$

TABLE II.—TABLE OF KEYSTONES FOR ARCHES OF FIRST-CLASS CUT-STONE.

Span in feet.	Rise in parts of the span.						
	½	⅓	¼	⅕	⅙	⅛	⅒
	key. ft.	key. ft.	key. ft.	key. ft.	key. ft.	key. ft.	key. ft.
2	0.55	0.56	0.58	0.60	0.61	0.64	0.68
4	0.70	0.72	0.74	0.76	0.79	0.83	0.88
6	0.81	0.83	0.86	0.89	0.92	0.97	1.03
8	0.91	0.93	0.96	1.00	1.03	1.09	1.16
10	0.99	1.01	1.04	1.07	1.11	1.18	1.26
15	1.17	1.19	1.22	1.26	1.30	1.40	1.50
20	1.32	1.35	1.38	1.43	1.48	1.59	1.70
25	1.45	1.48	1.53	1.58	1.64	1.76	1.88
30	1.57	1.60	1.65	1.71	1.78	1.91	2.04
35	1.68	1.70	1.76	1.83	1.90	2.04	2.19
40	1.78	1.81	1.88	1.95	2.03	2.18	2.33
50	1.97	2.00	2.08	2.16	2.25	2.41	2.58
60	2.14	2.18	2.26	2.35	2.44	2.62	2.80
80	2.44	2.49	2.58	2.68	2.78	2.98	3.18
100	2.70	2.75	2.86	2.97	3.09	3.32	3.55
120	2.94	2.99	3.10	3.22	3.35	3.61	3.88
140	3.16	3.21	3.33	3.46	3.60	3.87	4.15
160	3.36	3.44	3.58	3.72	3.87	4.17	
180	3.56	3.63	3.75	3.90	4.06	4.38	
200	3.74	3.81	3.95	4.12	4.29		
220	3.91	4.00	4.13	4.30	4.48		
240	4.07	4.15	4.30	4.48			
260	4.23	4.31	4.47	4.66			
280	4.38	4.46	4.63				
300	4.53	4.62	4.80				

Trautwine's Rule would give nearly the same, or

$$\frac{\sqrt{10+10}}{4}+0.2 \text{ foot}=1.3 \text{ feet.}$$

But if we should compute the stability of a semicircular arch of 20-foot span and 1.3-foot depth of keystone, we should find that the arch was very unstable; hence, in this case, we must throw the rule aside and go by our own judgment. In the opinion of the author, such an arch should have at least 2½ feet depth of arch-ring, and we will try the stability of the arch with that thickness.

In all calculations on the arch, it is customary to consider the arch to be one foot thick at right angles to its face; for it is evident that if an arch one foot thick is stable, any number of arches of the same dimensions built alongside of it would be stable.

Graphic Solution of the Stability of the Arch.—The most convenient method of determining the stability of the arch is by the graphic method, as it is called.

1st Step.—Draw one-half the arch to as large a scale as convenient and divide it up into voussoirs of equal size. In this example, shown in Fig. 3, we have divided the arch-ring into ten equal voussoirs. (It is not necessary that these should be the actual voussoirs of which the arch is built.) The next step is to find the area of each voussoir. Where the arch-ring is divided into voussoirs of equal size, this is easiest done by computing the area of the arch-ring and dividing by the number of voussoirs.

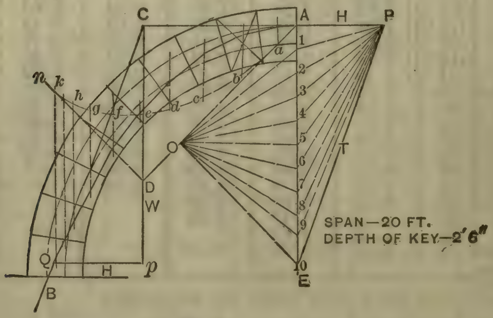


Fig. 3

Rule for area of one-half of arch-ring is as follows:

Area in square feet $=0.7854\times$(outside radius squared $-$ inside radius squared).

In this example the whole area equals $0.7854\times(12.5^2-10^2)=44.2$ square feet. As there are ten equal voussoirs, the area of each voussoir is 4.4 square feet.

Having drawn out one-half of the arch-ring, we divide each joint into three equal parts; and from the point A (Fig. 3) we lay off to a scale the area of each voussoir, one below the other, commencing with the top voussoir. The whole length of the line AE will equal the whole area drawn to same scale.

The next step is to find the vertical line passing through the centre of gravity of the whole arch-ring. To do this, it is first necessary to draw vertical lines through the centre of gravity of each voussoir. The centre of gravity of one voussoir may be found by the method of diagonals, as in the second voussoir from

the top (Fig. 3). Having the centre of gravity of one voussoir, the centres of gravity of the others can easily be obtained from it.

Next, from A and E (Fig. 3) draw lines at 45° with AE, intersecting at O. Draw $O1$, $O2$, $O3$, etc. Then, where AO intersects the first vertical line at a, draw a line parallel to $O1$, intersecting the second vertical at b. Draw bc parallel to $O2$, cd parallel to $O3$, and so on to kn parallel to $O10$; prolong this line downward until it intersects AO, prolonged, at D. Then a vertical line drawn through D will pass through the centre of gravity of the arch-ring.

2D STEP.—Draw a horizontal line through A (the upper part of the middle third), and a vertical line through D, the two lines intersecting at C (Fig. 3).

Now, that the arch shall be stable, it is considered necessary that it shall be possible to draw a line of resistance of the arch within the middle third. We will, then, first assume that the line of resistance shall act at A and come out at B.

Then draw the line CB, and a horizontal line opposite the point 10, between Q and p. This horizontal line represents the horizontal thrust at the crown.

Draw AP equal to Qp, and the lines $P1$, $P2$, $P3$, etc.

Then, from the point where AC prolonged intersects the first vertical, draw a line to the second vertical parallel to $P1$; from this point a line to the third vertical parallel to $P2$; and so on. The last line should pass through B. If these lines, which we will call the line of resistance, all lie within the middle third, the arch may be considered to be stable. Should the line of resistance pass outside of the arch-ring, the arch should be considered unstable. In Fig. 3 this line does not all lie in the middle third, and we must see if a line of resistance can yet be drawn within that limit.

2D TRIAL.—The line of resistance in Fig. 3 passes farthest from the middle third at the seventh joint from the top; and we will next pass a line of resistance through A and where the lower line of the middle third cuts the seventh joint, or at D (Fig. 4).

To do this we must prolong the line gh, parallel to $O7$ (Fig. 4), until it intersects AO. In this case it intersects it at O, but this is merely a coincidence; it would not always do so. Through O draw a vertical intersecting PA prolonged at C. Draw a line through C and D, and the horizontal line pQ, opposite the point 7; this line represents the new horizontal thrust H_1. Draw $AP = pQ$, and the lines $P1$, $P2$, etc.; then draw the line of resistance

as before. It should pass through D if drawn correctly. This time we see that the line of resistance lies within the middle third, except for just a short distance at the springing; and hence we may consider the arch stable. If it had gone outside the middle third this time, to any great extent, we should have considered the arch unstable.

The above is the method of determining the stability of an unloaded semicircular arch. Such a case very seldom occurs in practice; but it is a good example to illustrate the method, which applies to all other cases, with a little difference in the method of determining the centre of gravity of loaded arches.

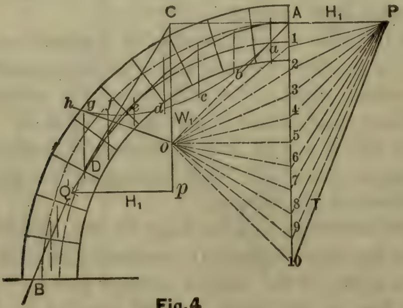


Fig.4

EXAMPLE II.—*Loaded or surcharged semicircular arch.*

We will take the same arch as in Example I. and suppose it to be loaded with a wall of masonry of the same thickness and weight per square foot as that of the arch-ring, the horizontal surface of the wall being 3 feet 6 inches above the arch-ring at the crown.

1st STEP.—*Find centre of gravity.*

Commencing at the crown, divide the load and arch-ring into strips two feet wide, making the last strip the width of the arch-ring at the springing. Then draw the joints as shown in Fig. 5. Measure with the scale the length of each vertical line, Aa, Bb, etc.; then the area of $AaBb$ is equal to the length of $Aa+Bb$, as the distance between them is just two feet. The area of $FfKk$ is, of course, $Ff \times$ width of arch-ring.

In this case the areas of the slices are as shown by the figures on their faces (Fig. 5).

Now divide the arch-ring into thirds, and from the top of the

middle third, at R, lay off in succession, to a scale, the areas of the slices, commencing with the first slice from the crown, $AaBb$. These areas, when measured off, will be represented by the line $R1, 2, 3 \ldots 6$ (Fig. 5). From the extremities of this line, R and 6, draw lines at 45° with a vertical, intersecting at O. From O draw lines to 1, 2, 3, 4, 5, and 6. Next draw a vertical line through the centre of each slice (these lines, in Fig. 5, are numbered 1, 2, 3, etc.). From the point in which the line RO intersects vertical 1, draw a line parallel to $O1$, to the line 2. From this point draw a line to vertical 3, parallel to $O2$, and so on. The line parallel to $O5$ will intersect vertical 6 at Y. Then through Y draw a line downwards at 45°, intersecting OR at X. A vertical line drawn through X will pass through the centre of gravity of the arch-ring and its load.

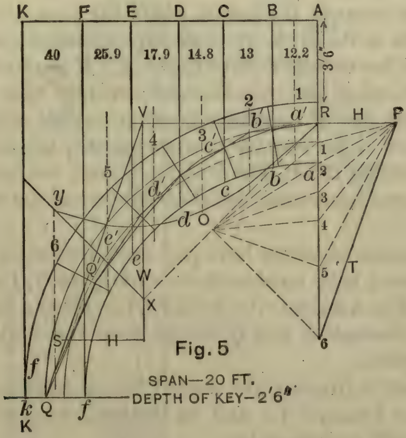


Fig. 5

2D STEP.—*To find the thrust at the crown and at the springing.* To find the thrust at the crown, draw a vertical line through X, and a horizontal line through R, intersecting at V. Now, the weight of arch and load, and the resultant thrust of arch, must act through this point. We will also make the condition that the thrust shall pass through Q, the outer edge of the middle third. Then the thrust of the arch must act in the line VQ. Opposite 6, on the vertical line through R, draw a horizontal line H, between VX and VQ. This horizontal line represents a hori-

zontal thrust at R, which would cause the resultant thrust of the arch to pass through Q. Now draw the horizontal line RP, equal in length to H, and from P draw lines 1, 2, 3 . . . 6. The line $P6$ represents the thrust of the arch at the springing. Its amount in cubic feet of masonry can be determined by measuring its length to the proper scale.

3D STEP.—*To draw the line of resistance.*

The lines $P1, P2, P3$, etc., represent the magnitude and direction of the thrust at each joint of the arch. Thus $P1$ represents the thrust of the first voussoir and its load, $P2$ that of the first two voussoirs and their loads, and so on. Then from the point a', where the line RP, prolonged, intersects the vertical line 1, draw a line $a'\,b'$ parallel to $P1$; from b', on 2, draw a line $b'c'$ parallel to $P2$, and so on. The last line should pass through Q and be parallel to $P6$.

Now, if we connect the points where the lines $a'b'$, $b'c'$, etc., cut the joints of the arch, we shall have a broken line, which is known as the line of resistance of the arch. If this line lies within the middle third of the arch, then we conclude that the arch is stable. If the line of resistance goes far outside of the middle, we must see if it be possible to draw another line of resistance within the middle third; and if, after a trial, we find that it is not possible, we must conclude that the arch is not safe, or unstable.

In the example which we have just been discussing, the line of resistance goes a little outside of the middle third; but it is very probable that on a second trial we should find that a line of resistance passed through R and Q' would lie almost entirely within the middle third.

The method of drawing the second line of resistance was explained under Example I.; and, as the same method applies to all cases, we will not repeat it.

The method given for Example II. would apply equally well for a semi-elliptical arch.

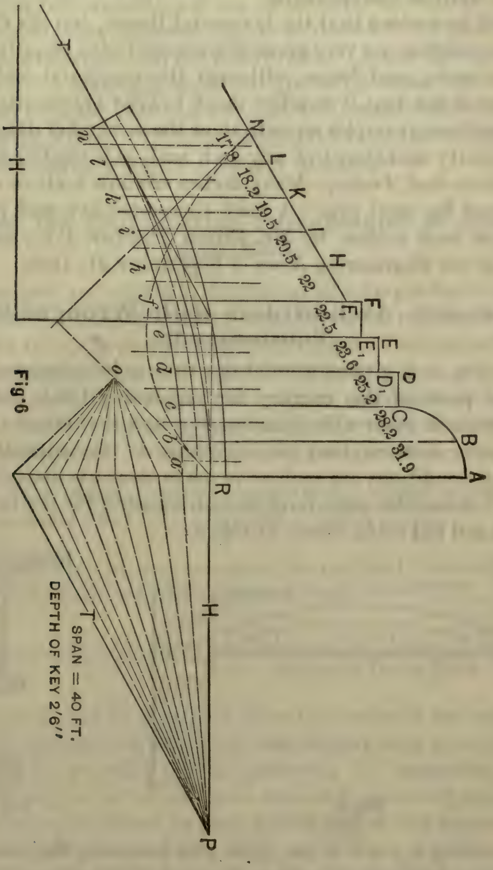
EXAMPLE III.—*Segmental arch, with load* (Fig. 6).

1ST STEP.—*To determine the centre of gravity.*

In this case we proceed, the same as in the latter, to divide the arch-ring and its load into vertical slices two feet wide, and compute the area of the slices by measuring the length of the vertical lines Aa, Bb, etc. Having computed the areas of the slices, we lay them off in order from R to a convenient scale and then proceed exactly as in Example II., the remaining steps to determine

the thrust and the lines of resistance are also the same as given under Example II.

In a flat segmental arch there is practically no need of dividing the arch-ring into voussoirs by joints radiating from a centre, but



to consider the joints to be vertical. Of course, when built, they must be made to radiate.

Fig. 6 shows the computation for an arch of 40-foot span and with a load $13\frac{1}{2}$ feet high at the centre. The depth of the arch-ring is 2 feet 6 inches.

It will be seen that the curve of pressures lies entirely within the middle third, and hence the arch is abundantly safe, or stable. It should be remarked that the line of resistance in a segmental arch should be drawn through the *lower* edge of the middle third at the springing.

It will be noticed that the horizontal thrust, and the thrust T, at the springing, are very great as compared with those in a semicircular arch; and hence, although the segmental arch is the stronger of the two, it requires much heavier abutments.

These three examples serve to show the method of determining the stability and thrust of any arch such as is used in building.

Concrete-Steel Arches.—Many arches are now built of concrete reinforced by steel ribs. A very comprehensive and practical paper on such arches, by Mr. Edwin Thacher, C.E., was published in the *Engineering News* of September 21, 1899.

Cast-iron Arch-girders with Wrought-iron Tension-rods.

Cast-iron arch-girders were at one time quite extensively used in some parts of this country for supporting brick walls over store fronts or other wide openings, but now that structural steel has become so cheap they are seldom used. Occasionally, however, the conditions are such as to make their use desirable.

Fig. 7 shows the usual form of such a girder, the section of the casting and rod being shown in Fig. 8.

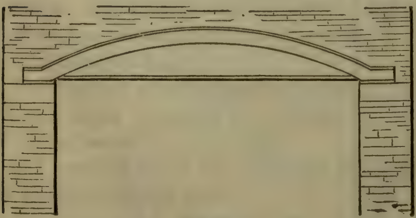


Fig. 7

Fig. 8

The casting is made in one piece with box ends, the latter having grooves and seats to receive the wrought-iron tie-rod.

The tie-rod is made from one-eighth to three-eighths of an inch shorter than the casting, and has square ends forming shoulders so as to fit into the castings. The rod has usually one weld on its length, and great care should be taken that this weld be perfect.

The rod is expanded by heat, and then placed in position in the casting, and allowed to contract in cooling; thus tying the two ends of the casting together to form abutments for receiving the horizontal thrust of the arch. If the rod is too long, it will not receive the full proportion of the strain until the cast-iron has so far deflected, that its lower edge is subjected to a severe tensile strength, which cast-iron can feebly resist. If the tie-rod is made too short, the casting is cambered up, and a severe initial strain put upon both the cast and wrought iron, which enfeebles both for carrying a load. The girders should have a rise of about two feet six inches on a length of twenty-five feet.*

Rules for Calculating Dimensions of Girder and Rod.

A cast-iron arch-girder is considered as a long column, subject to a certain amount of bending-strain; and the resistance will be governed by the laws affecting the strength of beams, as well as by those relating to the strength of columns.

If we regard the arch as flexible, or as possessing no inherent stiffness, and the rod as a chord without weight, the horizontal thrust or stress will be represented by the formula.

$$\frac{\text{Hor. thrust}}{\text{or stress (lbs.)}} = \frac{\text{load on girder in lbs.} \times \text{span in feet}}{8 \times \text{rise of girder in feet}}. \quad (1)$$

From this rule we can calculate the required diameter of the tension-rod, which may be expressed thus

$$\text{Diameter in inches} \dagger = \sqrt{\frac{\text{load on girder} \times \text{span in feet}}{8 \times \text{rise in feet} \times 7854}}. \quad (2)$$

The rise should be measured from the centre of the rod to the centre of the cast-iron arch; the load should be in pounds.

The rule generally used, however, in proportioning the wrought-iron tie to the cast-iron arch is *to allow one square inch of cross-section of tie-rod for every ten net tons of load imposed upon* the span of the arch.

The following table, taken from Mr. Fryer's book on "Architectural Iron-Work," shows the *section of the cast-iron arch re-*

* "Architectural Iron-Work for Buildings," William J. Fryer, Jr., p. 38.

† For a working stress of 10000 lbs. per square inch. For rods of mild steel, use 9800 in place of 7854.

quired to support solid brick walls, and having a span of from 13 *to* 26 *feet.*

Height of wall.	Thickness of wall.	Dimensions of section.		
		Top flange.	Centre web.	Bulb.
40 feet	12 inches	12″×1″	12″×¾″	3″ ×2″
50 "	12 "	12″×1¼″	12″×⅞″	3″ ×2″
40 "	16 "	12″×1¼″	12″×⅞″	3½″×2″
50 "	16 "	16″×1½″	12″×1″	4″ ×2″

EXAMPLE I.—It is desired to support a 12-inch brick wall 40 feet high over an opening 20 feet wide, with a cast-iron arch-girder. What should be the dimensions of the girder?

For the casting, we find from the table that the cross-section of the flange should be 12 inches by 1 inch; of the web, 12 inches by ¾ inch; and of the bulb, 3 inches by 2 inches. We will make the rise of the girder 2 feet and 6 inches, and from Formula 2 we find *

$$\text{Diam. of rod in inches} = \sqrt{\frac{\text{weight of wall} \times \text{span}}{8 \times \text{rise of arch in feet} \times 7854}}$$

$$= \sqrt{\frac{(20 \times 20 \times 112) \times 20}{8 \times 2\tfrac{1}{2} \times 7854}} = \sqrt{5.7} = 2\tfrac{3}{8} \text{ ins.}$$

Figs. 9 and 10 show a cast-iron arched girder that was used to

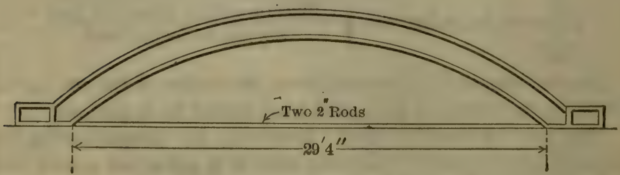
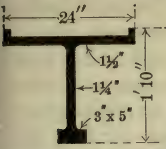


Fig. 9

support a central tower over the crossing of the nave and transept

* Considering that the girder would only support about twenty feet of the wall in height, the wall above that supporting itself. See "Beams Supporting Brick Walls," Chapter XV.

of St. John's Church, Stockton, California,



SECTION AT CENTER.
Fig. 10

Mr. A. Page Brown, architect. The clear span is 29½ feet, and the height of the wall above the girder 18 feet. One object in using such a girder in this place was to get the height in the centre without also raising the supports, which could not be obtained with a steel plate girder. The church has a vaulted ceiling which comes just below the arch of the girder, the tie-rod being exposed.

The rise of the casting in this case is rather more than common, the usual rise being from $\frac{1}{10}$ to $\frac{1}{8}$ of the span.

Fig. 11 shows an arch girder, with the casting in three sections, with pin joints, the head of the suspending pieces *AA*, forming the pin. The flexible bolt connections serve to hold the sections in place laterally.

Mr. Peter H. Jackson, C.E., of San Francisco, who has had an extended experience with this form of construction, says of it: "The Franklin girder is unequaled for strength in the application of the same amount of metal in any other form. This cast-iron arch is in effect a device for employing cast iron compressively, neutralizing the tensile strain due to transverse strain. It has been favorably commented on by the scientific papers of the United States, London and Paris."

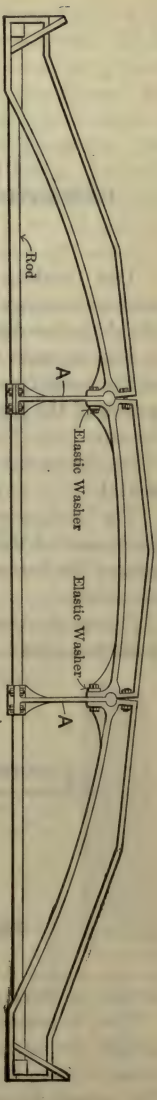


Fig. 11

CHAPTER IX.

BENDING-MOMENTS AND SUPPORTING FORCES.

THE bending-moment of a beam or truss represents the destructive energy of the load on the beam or truss at any point for which the bending-moment is computed.

The moment of a force around any given axis is the product of the force into the perpendicular distance between the line of action of the force and the axis, or the product of the force into its arm.

In a beam the forces or loads are all vertical and the arms horizontal.

The bending-moment at any cross-section of a beam is the algebraic sum of the moments of the forces tending to turn the beam around the horizontal axis passing through the centre of gravity of the section.

EXAMPLE.—Suppose we have a beam with one end securely fixed into a wall and the other end projecting from it, as in Fig. 1.

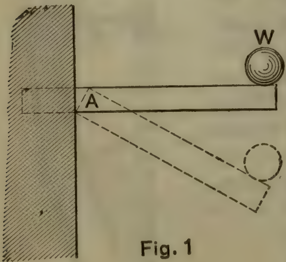


Fig. 1

Let us now suppose we have a weight which if placed at the end of the beam will cause it to break at the point of support.

Then, if we were to place the weight on the beam at a point near the wall, the beam would support the weight easily; but as we move the weight towards the outer end of the beam, the beam bends more and more, and when the weight is at the end, the beam breaks, as shown by the dotted lines, Fig. 1.

Now, it is evident that the destructive energy of the weight is greater the farther the weight is removed from the wall-end of the beam, although the weight itself remains the same all the time. The reason for this is that the moment of the weight tends to

turn the beam about the point A, and thus produces a pull on the upper fibres of the beam and compresses the lower fibres. As the weight is moved out on the beam, its moment becomes greater, and hence also the pull and compression on the fibres; and when the moment of the weight produces a greater tension or compression on the fibres than they are capable of resisting, they fail and the beam breaks. Before the fibres break, however, they commence to stretch, and this allows the beam to bend; hence the name "bending-moment" has been given to the moment which causes a beam to bend, and perhaps ultimately to break.

There may, of course, be several loads on a beam, and each one having a different moment tending to bend the beam; and it may also occur that some of the weights may tend to turn the beam in different directions; the algebraic sum of their moments (calling those tending to turn the beam to the right + and the others −) would be the bending-moment of the beam.

Knowing the bending-moment of a beam, we have only to find the section of the beam that is capable of resisting it, as is shown in the general theory of beams, Chapter XV.

To determine the bending-moments of beams mathematically requires considerable training in mechanics and mathematics; but, as most beams may be placed under some one of the following cases, we shall give the bending-moment for these cases and then show how the bending-moment for any other methods of loading may be easily obtained by a scale diagram.

Examples of Bending-Moments.

Case I.

Beam fixed at one end and loaded with concentrated load W.

Bending-moment = $W \times L$. (L may, or may not, be the whole length of the beam according to where the weight is located.)

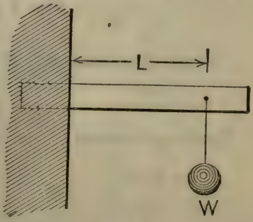
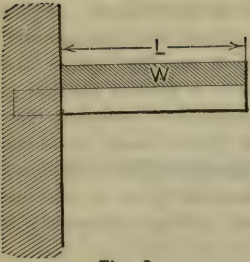


Fig. 2

Case II.

Beam fixed at one end, loaded with a distributed load W.



$$\text{Bending-moment} = W \times \frac{L}{2}.$$

Note.—If L is in feet, the bending-moment will be foot-pounds; if L is in inches, the bending-moment will be in inch-pounds. See general formula for beams, Chapter XV.

Fig. 3

Case III.

Beam fixed at one end, loaded with both a concentrated and a distributed load.

$$\text{Bending-moment} = P \times L_2 + W \times \frac{L_1}{2}.$$

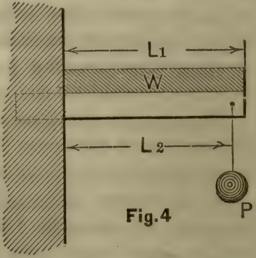
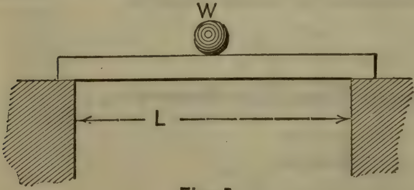


Fig. 4

Case IV.

Beam supported at both ends, loaded with concentrated load at centre.



$$\text{Bending-moment} = W \times \frac{L}{4}.$$

Fig. 5

Case V.

Beam supported at both ends, loaded with a distributed load W.

$$\text{Bending-moment} = W \times \frac{L}{8}.$$

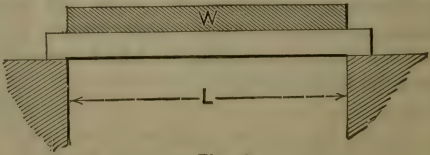
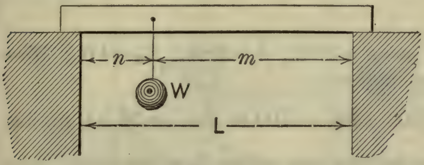


Fig. 6

Case VI.

Beam supported at both ends, loaded with concentrated load not at centre.



Bending-moment $=W\times\frac{m\times n}{L}$.

Fig. 7

Case VII.

Beam supported at both ends, loaded with two equal concentrated loads, equally distant from the centre.

Bending-moment $=W\times m$.

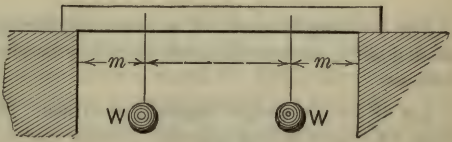
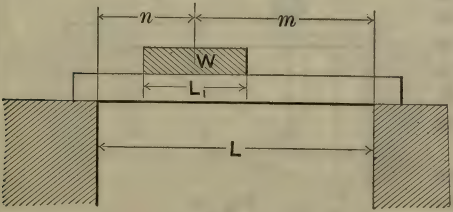


Fig. 8

From these examples it will be seen that all the quantities which enter into the bending-moment are the weight, the span, and the distance of point of application of concentrated load from each end.

Case VIII.

Beam supported at both ends, loaded with a distributed load over only a portion of the span.



Bending-moment $=\frac{W\times m\times n}{L}-\frac{W\times L_1}{8}$.

Fig. 9

When m and n are equal, bending-moment $=\frac{W\times L}{4}-\frac{W\times L_1}{8}$.

EXAMPLE.—Let $W=800$ lbs.; $n=8'$; $m=12'$; $L=20'$; $L_1=8'$. Then bending-moment =

$\frac{800\times 8\times 12}{20} - \frac{800\times 8}{8} = 36{,}480 - 800 = 3040$ foot-pounds or 36,480 inch-pounds.

EXAMPLE II.—Let $n=m=10$ ft; $L=20$ ft.; $L_1=4$ ft.; $W=600$ lbs. Then bending-moment =

$\frac{600\times 20}{4} - \frac{600\times 4}{8} = 3000-300 = 2700$ ft. lbs. or 32,400 in.-lbs.

The bending-moment for any case other than the above may easily be obtained by the graphic method, which will now be explained.

Graphic Method of Determining Bending-Moments.

The bending-moment of a beam supported at both ends and loaded with one concentrated load may be shown graphically, as follows:

Let W be the weight applied, as shown. Then, by rule under Case VI., the bending-moment directly under

$$W = W\times\frac{m\times n}{L}.$$

Draw the beam, with the given span, accurately to scale, and measure down the line AB (to a scale of pounds to the inch) equal to the bending-moment. Connect B with each end of the beam. If, then, we wished to find the bending-moment at any other point of the beam, as at o, draw the vertical line y to BC, and its length, measured to the same scale as AB, will give the bending moment at o.

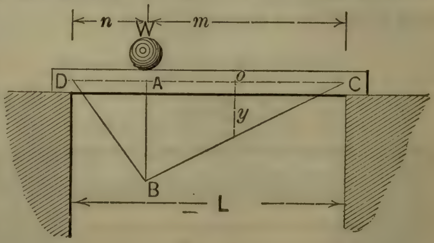


Fig. 10

Beam with two concentrated loads. (Fig. 11.)

To draw the bending-moment for a beam with two connected loads, first draw the dotted lines ABD and ACD, giving the out-

line of the bending-moment for each load separately; EB being equal to $W \times \frac{m \times n}{L}$ and FC equal to $P \times \frac{r \times s}{L}$.

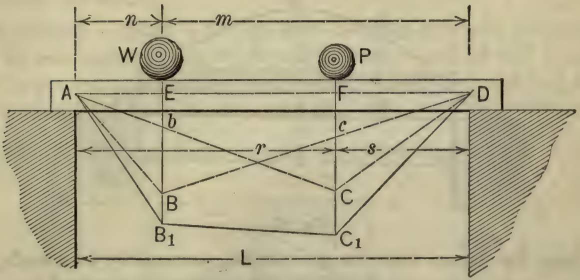


Fig. 11

Now, the bending-moment at the point E equals EB, due to the load W, and Eb, due to the load P; hence the bending-moment at E should be drawn equal to $EB+Eb=EB_1$; and at F the bending-moment should equal $FC+Fc=FC_1$. The outline for the bending-moment due to both loads, then, would be the line AB_1C_1D and the greatest bending-moment would, in this particular case, be FC_1.

Beam with three concentrated loads. (Fig. 12.)

Proceed as in the last case, and draw the bending-moment for each load separately. Then make $AD=A1+A2+A3$,

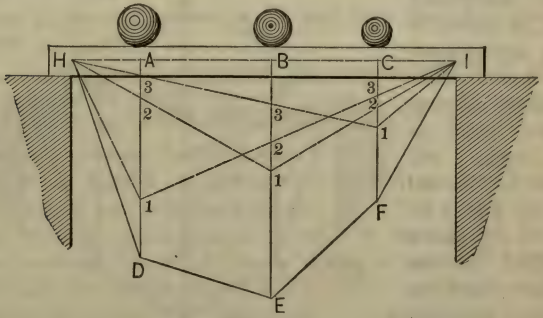


Fig. 12

$BE=B1+B2+B3$, and $CF=C1+C2+C3$. The line $HDEFI$ will then be the outline for the bending-moment due to all the

weights. The bending-moment for a beam loaded with any number of concentrated weights may be drawn in the same way.

Beam with uniformly distributed load. (Fig. 13.)

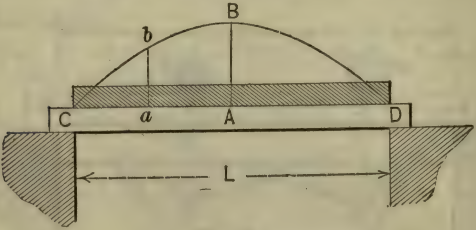


Fig. 13

Draw the beam with the given span, accurately to a scale, as before, and at the middle of the beam draw the vertical line AB equal to $W \times \frac{L}{8}$, W representing the whole distributed load. Then connect the points C, B, D by a parabola and it will give the outline of the bending-moments. If, now, we wanted the bending-moment at the point a, we have only to draw the vertical line ab, and measure it to the same scale as AB, and it will be the moment desired. Methods for drawing the parabola may be found in "Geometrical Problems," Part I.

Beam loaded with both distributed and concentrated loads. (Fig 14.)

To determine the bending-moment in this case, we have only to combine the methods for concentrated loads and for the distributed load, as shown in the accompanying figure. The bending-moment at any point on the beam will then be limited by the line ABC on top and $CDEFA$ on the bottom; and the greatest bending-moment will be the longest vertical line that can be drawn between these two bounding lines.

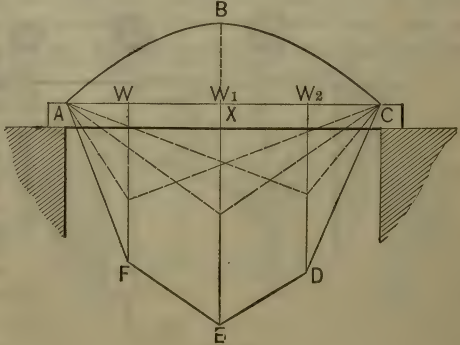


Fig. 14

For example, the bending-moment at X would be BE. The position of the greatest bending-moment will depend upon the position of the concentrated loads, and it may and may not occur at the centre.

EXAMPLE.—What is the greatest bending-moment in a beam of 20 feet span, loaded with a distributed load of 800 pounds and a concentrated load of 500 pounds 6 feet from one end, and a concentrated load of 600 pounds 7 feet from the other end?

Ans. 1st. The moment due to the distributed load is $W \times \frac{L}{8}$, or $\frac{800 \times 20}{8} =$ 2000 pounds. We, therefore, lay off to a scale, say 4000 pounds to the inch, $B1 = 2000$ pounds, and draw a parabola between the points A, B, and C.

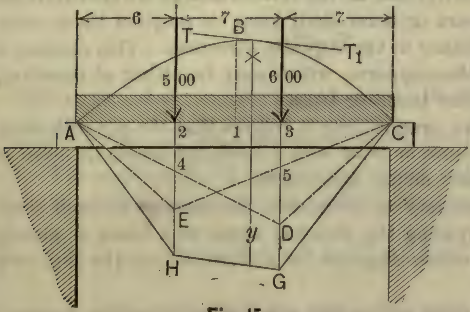


Fig. 15

2d. The bending-moment for the concentrated load of 500 pounds is $\frac{500 \times 6 \times 14}{20}$, or 2100 pounds. Hence we draw $E2 = 2100$ pounds to the same scale as $B1$, and then draw the lines AE and CE.

3d. The bending-moment for the concentrated load of 600 pounds is $\frac{600 \times 7 \times 13}{20}$, or 2730 pounds; and we draw $D3 = 2730$ pounds and connect D with A and C.

4th. Make $EH = 2 - 4$, and $DG = 3 - 5$, and connect G and H with C and A and with each other.

The greatest bending-moment will be represented by the longest vertical line which can be drawn between the parabola ABC and the broken line $AHGC$. In this example we find the longest vertical line which can be drawn is xy; and by scaling it we find the greatest bending-moment to be 5550 *foot-pounds*, applied 10 feet 11 inches from the point A.

In this case the position of the line Xy was determined by

drawing the line TT_1 parallel to HG, and tangent to ABC. The line Xy is drawn through the point of tangency.

NOTE.—If we wish the bending-moment in *inch-pounds*, multiply the moment in foot-pounds by 12.

Supporting Forces.

It is a fundamental principle of mechanics that for a body to be in equilibrium the forces acting upon it must balance each other. When, therefore, a body, such as a beam, girder, or truss, is subjected to loads acting downwards in a vertical direction, that the beam or truss shall keep its position there must be an equal resistance in the opposite direction. This resistance is furnished by the supports, which may be either of masonry, columns, or another beam or truss.

From the above propositions it follows that *the supports must react against the beam or truss with a combined resistance equal to the sum of the loads acting downwards.*

It is often necessary to determine the amount of each reaction, and in computing the shearing stress in a beam or girder, or in drawing a strain diagram for a truss, this is the first step of the problem.

The following rules will enable one to determine the supporting forces or reactions, for any manner of loading, when the beam, girder, or truss is supported at both ends.

These rules apply either to a beam, girder, or truss, and to any style of truss.

1. *When the loads are symmetrically disposed between the supports, each supporting force is equal to one-half of the total load.*

2. *For a single concentrated load applied at any point, as in Fig.* 16,

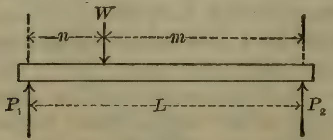


Fig. 16

$$P_1 = \frac{W \times m}{L}; \quad P_2 = W - P_1. \tag{1}$$

3. *For a distributed load applied over only a portion of the span,*

as in Fig. 17, assume the load to be concentrated at its centre, and use formula (1).

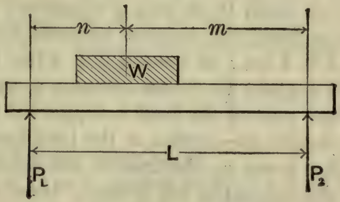


Fig. 17

EXAMPLE.—Let $n=8$, $m=12$, and $W=800$; then $P_1=\frac{800\times12}{20}=480$. When n and m are equal, then $P_1=P_2=\frac{1}{2}W$.

4. *For any number of concentrated forces, indicate the distances from the right-hand support, as in Fig.* 18; then

$$P_1=\frac{W_1m+W_2n+W_3o+W_4r+W_5s+W_6t}{L}; \tag{2}$$

$$P_2=W_1+W_2+W_3+W_4+W_5+W_6-P_1.$$

The same result would be obtained by finding the reaction of P_1 for each load by formula (1) and adding the reactions.

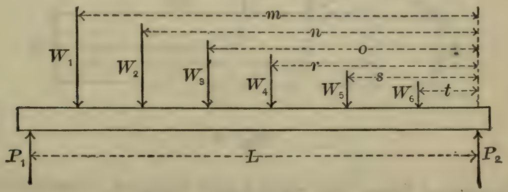


Fig. 18

When a truss is loaded unsymmetrically, the supporting forces will be found by formula (2), keeping the same notation, and using the same number of terms in the formula as there are loads on the truss.

It should always be borne in mind *that the sum of the reactions is equal to the sum of the loads.*

If the beam or girder supports a distributed load and also concentrated loads, to each of the reactions obtained by formulas (1) or (2) add one-half of the distributed load.

EXAMPLE 1.—A beam 16 ft. long is supported at each end by the tie beam of a truss, the distance between the centres of the trusses being 15 ft. 4 ins.; at a point 4 ft. 4 ins. from the centre of the left-hand support the beam sustains a concentrated load of 12 tons. What part of the load is supported by each truss?

Ans. Let P_1 denote the portion of the load borne by the truss at the left, P_2 that borne by the truss at the right, and L the distance between the centres of the trusses; then, by formula (1), we have

$$P_1=\frac{12\times 11}{15\frac{1}{3}}=8.6 \text{ tons} \quad \text{and} \quad P_2=12-8.6=3.4 \text{ tons}.$$

EXAMPLE 2.—A girder of 30 feet span is loaded with a distributed load of 15 tons and with six concentrated loads of 5, 6, 4, 8, 3, and 2 tons, arranged consecutively from left to right.

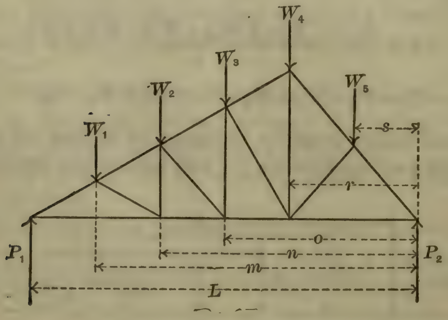


Fig. 19

The distances from the right-hand support, corresponding to those in Fig. 18, are: $m=28$, $n=22$, $o=16$, $r=12$, $s=10$, $t=6$. What will be the reaction at each support?

Ans. First find reactions for concentrated loads. By formula (2)

$$P_1=\frac{5\times 28+6\times 22+4\times 16+8\times 12+3\times 10+2\times 6}{30}=15.8 \text{ tons};$$

$$P_2=5+6+4+8+3+2-15.8=12.2 \text{ tons}.$$

One-half of the distributed load is 7.5 tons; then P_1 for whole load $=15.8+7.5=23.3$ tons and $P_2=19.7$ tons.

The reactions obtained for the concentrated loads may be verified by multiplying each load by its distance from the other support and dividing the sum of the products by the span. The result will be P_2. Thus in the above example

$$P_2=\frac{5\times2+6\times8+4\times14+8\times18+3\times20+2\times24}{30}=12.2,$$

which proves the former calculation.

Example 3.—A truss is loaded in such a way that the loads correspond to 3 tons for W_1 (Fig. 19), 3 tons for W_2, 5 tons for W_3, 6 tons for W_4, and 5 tons for W_5; the distance L is 48 feet; m, 40 feet; n, 32 feet; o, 24 feet; r, 16 feet, and s, 8 feet. What is the reaction at each support?

$$\textit{Ans.}\ P_1=\frac{3\times40+3\times32+5\times24+6\times16+5\times8}{48}=9.83 \text{ tons};$$

$$P_2=3+3+5+6+5-9.83=12.17 \text{ tons}.$$

CHAPTER X.

MOMENTS OF INERTIA AND RESISTANCE, AND RADIUS OF GYRATION.

DIMENSIONS AND PROPERTIES OF STRUCTURAL SHAPES.

Moment of Inertia.

The strength of sections to resist strains, either as girders or as posts, depends not only on the area, but also on the form of the cross-section. The property of the section which represents the effect of the form upon the strength of a beam or post is its moment of inertia, usually denoted by I. *The moment of inertia for any cross-section is the sum of the products obtained by multiplying the area of each particle in the cross-section by the square of its distance from the neutral axis.*

The neutral axis of a beam is the line on which there is neither tension nor compression, and when the stresses are within the elastic limit of the material, it can be shown that the neutral axis passes through the centre of gravity of the cross-section.

For most forms of cross-section the moment of inertia is best found by the aid of the calculus; though it may be obtained by dividing the figure into small squares or triangles, and multiplying their areas by the squares of the distance of their centres of gravity from the neutral axis. The sum of all the products will be the moment of inertia of the section.

Moment of Resistance.

The resistance of a beam to bending and cross-breaking at any given cross-section is the moment of the two equal and opposite forces, consisting of the thrust along the longitudinally compressed layers, and the tension along the longitudinally stretched layers.

This moment, called the "moment of resistance," is, for any given cross-section of a beam, equal to

$$\left(\frac{\text{moment of inertia}}{\text{extreme distance from axis}}\right) \times \text{modulus of rupture or fibre stress.}$$

The moment of resistance forms a part of all formulas for the strength of beams. The portion of the above formula included in parenthesis is sometimes erroneously designated the moment of resistance; in the handbooks published by the manufacturers of structural steel shapes, it is now designated as the *section modulus*, and for the sake of uniformity, the author has adopted the same term.

Radius of Gyration.

The effect of the form or section of a *column* upon its strength is determined by a property called the radius of gyration. The value of the radius of gyration of any section is determined by the formula, $r=\sqrt{\text{moment of inertia} \div \text{section area}}$.

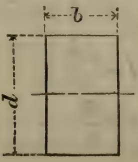
The moment of inertia and radius of gyration of a section are always taken about an axis passing through the centre of gravity of the section. For all sections except circles there will be at least two radii of gyration; the least of these will be that taken about the axis around which the column, strut, or beam is most likely to bend.

Formulas for the moment of inertia, radius of gyration, and section modulus of the principal elementary sections are given below. In the case of hollow or re-entering sections, the moment of the hollow portion is to be subtracted from that of the enclosing area.

Moments of inertia when referred to the same axis can be added or subtracted like any other qualities which are of the same kind.

Moments of Inertia, Section Modulus, and Radii of Gyration.

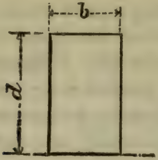
I = moment of inertia.
R = section modulus.
r = radius of gyration.
A = area of the section.
Position of neutral axis represented by broken line.



$$I=\frac{bd^3}{12}.$$

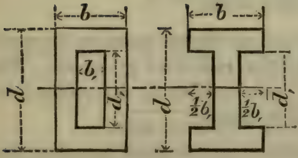
$$R=\frac{bd^2}{6}.$$

$$r^2=\frac{d^2}{12}.$$



$$I=\frac{bd^3}{3}.$$

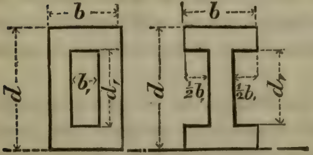
$$r^2=\frac{d^2}{3}.$$



$$I=\frac{bd^3-b_{,}d_{,}^3}{12}.$$

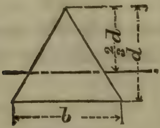
$$R=\frac{2I}{d}.$$

$$r^2=\frac{I}{bd-b_{,}d_{,}}.$$



$$I=\frac{bd^3}{3}-b_1d_1\frac{d^2}{4}-\frac{b_{,}d_{,}^3}{12}.$$

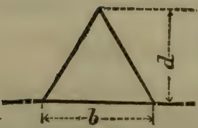
$$r^2=\frac{I}{A}.$$



$$I=\frac{bd^3}{36}.$$

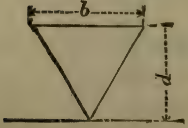
$$R=\frac{3I}{2d}=\frac{bd^2}{24}.$$

$$r^2=\frac{I}{A}=\frac{d^2}{18}.$$



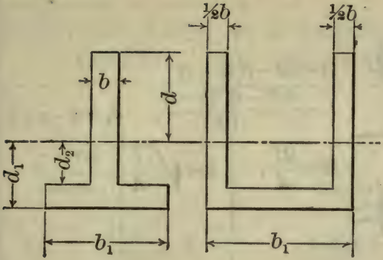
$$I=\frac{bd^3}{12}.$$

$$r^2=\frac{d^2}{6}.$$



$$I=\frac{bd^3}{4}.$$

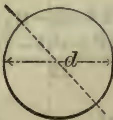
$$r^2=\frac{d^2}{2}.$$



$$I=\frac{bd^3+b_{,}d_{,}^3-(b_{,}-b)d_2^3\,*}{3}.$$

$$R=\frac{I}{d}.$$

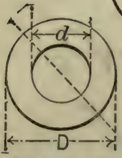
$$r^2=\frac{I}{A}.$$



$$I = 0\,0491d^4.$$

$$R=0.0982d^3.$$

$$r^2=\frac{d^2}{16}.$$

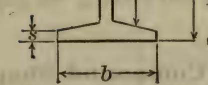


$$I = 0.0491(D^4-d^4).$$

$$R= 0.0982\left(D^3-\frac{d^4}{D}\right).$$

$$r^2=\frac{1}{16}\,\frac{D^4-d^4}{D^2-d^2}.$$

$$r = \frac{D+d}{5.64}$$ } (a very close approximation).

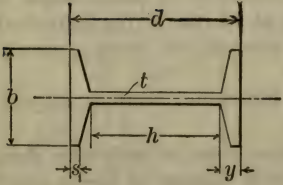


$$A=bd-d_1(b-t).$$

$$I =\frac{bd^3-(b-t)d_1^3}{12}.$$

$$R=\frac{2I}{d}. \qquad r=\sqrt{\frac{I}{A}}.$$

$$d_1=d-s+y.$$



$$I =\frac{b_3(s+y)+ht^3}{12}.$$

$$R=\frac{2I}{b}. \qquad r=\sqrt{\frac{I}{A}}.$$

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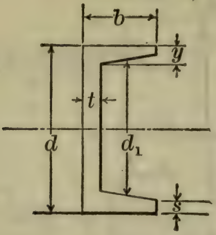
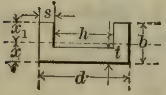
* To find the value of d, see page 240.



$$A = bd - d_1(b - t).$$

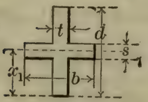
$$I = \frac{bd^3 - d_1^3(b - t)}{12}. \qquad d_1 = d - s + y$$

$$R = \frac{2I}{d}. \qquad r = \sqrt{\frac{I}{A}}.$$

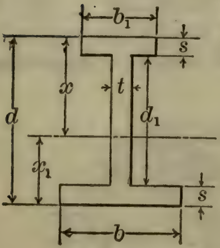


$$A = td + s(b - t).$$

$$I = \frac{td^3 + s^3(b - t)}{12}.$$



$$R = \frac{2I}{d}. \qquad r = \sqrt{\frac{I}{A}}.$$



$$A = b's + bs + d_1t.$$

$$I = \frac{b'x^3 - (b' - t) \times (x - s)^3}{3} + \frac{bx_1^3 - (b - t) \times (x_1 - s)^3}{3}$$

$$R = \frac{I}{x'}. \qquad r = \sqrt{\frac{I}{A}}.$$

To find x and x' see page 239.

Moments of Inertia of Compound Shapes.

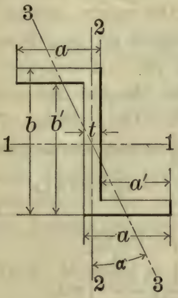
The moment of inertia of any combination of single shapes is equal to the sum of the moments of inertia of the individual shapes taken about an axis passing through the *centre of gravity* of the combination.

The moment of inertia of the individual sections may be obtained by the following rule:

The moment of inertia of any section about an axis other than through its centre of gravity is equal to its moment of inertia about a parallel axis passing through its centre of gravity plus the area of the section multiplied by the square of the distance between the axes.

Thus, the moment of inertia of the angle (Fig. 1) about the

FORMULAS USED FOR COMPUTING MOMENTS OF INERTIA FOR STANDARD SECTIONS.



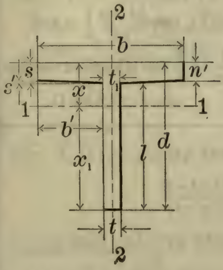
$$A=[b+2(a-t)]t.$$

$$\text{Tan } 2\alpha=-\frac{(bt-t^2)(a^2-at)}{I-I'}.$$

$$I,\ \text{Axis } 1-1=\frac{ab^3-a'(b-2t)^3}{12}.$$

$$I',\ \text{Axis } 2-2=\frac{b(a+a')^3-2a'^3b-6a'a^2b}{12}.$$

$$I''\ \text{minimum, Axis } 3-3=\frac{I'\cos^2\alpha-I\sin^2\alpha}{\cos 2\alpha}.$$

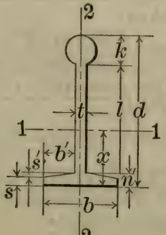


$$A=\frac{l(t+t_1)}{2}+n't_1+b'(s+n').$$

$$x=\frac{3s^2(b-t_1)+2b's'(s'+3s)+3t_1d^2-l(t_1-t)(3d-l)}{6A}$$

$$I,\ \text{Axis } 1-1=\frac{l^3(3t+t_1)+4bn'^3-2b's'^3}{12}-A(x-n')^2$$

$$I',\ \text{Axis } 2-2=\frac{sb^3+s't_1^3+lt^3}{12}+\frac{s'b'[2b'^2+(2b'+3t_1)^2]}{36}+\frac{l(t_1-t)[(t_1-t)^2+2(t_1+2t)^2]}{144}.$$



$e=$ area of head.

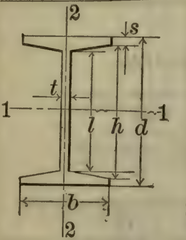
$$A=e+t(d-k)+(b-t)\left(s+\frac{s'}{2}\right).$$

$$x=\frac{e(2d-k)+t(d-k)^2+(b-t)\left(s^2+ss'+\frac{s'^2}{3}\right)}{2A}.$$

$$I,\ \text{Axis } 1-1=e\left[\frac{k^2}{16}+\left(d-\frac{2s+k}{2}\right)^2\right]+\frac{t(l+s')^3}{3}+\frac{b's'^3+2bs^3}{6}-A(x-s)^2.$$

$$I',\ \text{Axis } 2-2=\frac{ek^2}{16}+\frac{t^3(l+s')+sb^3}{12}+\frac{s'b'[2b'^2+(2b'+3t)^2]}{36}.$$

FORMULAS USED FOR COMPUTING MOMENTS OF INERTIA FOR STANDARD SECTIONS—*Continued.*



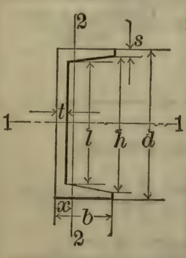
$$A = td + 2s(b-t) + \frac{(b-t)^2}{12}.$$

$$I, \text{ Axis } 1-1 = \frac{bd^3}{12} - \frac{h^4 - l^4}{8}.$$

$$I', \text{ Axis } 2-2 = \frac{b^3 s}{6} + \frac{lt^3}{12} + \frac{b^4 - t^4}{288}.$$

Slope of flange $= g = \dfrac{h-l}{b-t} = \dfrac{1}{6}$ for standard sections.

$h = d - 2s.$ $\qquad l = h - g(b-t).$



$$A = td + 2s(b-t) + \frac{(b-t)^2}{6}.$$

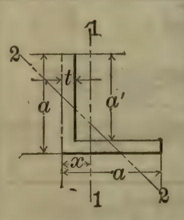
$$x = \left[b^2 s + \frac{ht^2}{2} + \frac{(b-t)^2(b+2t)}{18} \right] \div A.$$

$$I, \text{ Axis } 1-1 = \frac{bd^3}{12} - \frac{h^4 - l^4}{16}.$$

$$I', \text{ Axis } 2-2 = \frac{1}{3}\left[2sb^3 + lt^3 + \frac{b^4 - t^4}{12} \right] - Ax^2.$$

Slope of flange $= g = \dfrac{h-l}{2(b-t)} = \dfrac{1}{6}$ for standard sections.

$h = d - 2s.$ $\qquad l = h - 2g(b-t).$

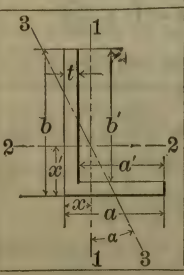


$$A = t(2a-t).$$

$$x = \frac{a^2 + at - t^2}{2(2a-t)}.$$

$$I, \text{ Axis } 1-1 = \frac{t(a-x)^3 + ax^3 - (a-t)(x-t)^3}{3}.$$

$$I'', \text{ Axis } 2-2 = \frac{2x^4 - 2(x-t)^4 + t\left[a - \left(2x - \frac{t}{2} \right) \right]^3}{3}$$



$$A = t(a+b-t).$$

$$x = \frac{t(2a'+b) + a'^2}{2(a'+b)}. \qquad x' = \frac{t(2b'+a) + b'^2}{2(b'+a)}.$$

$$\text{Tan } 2\alpha = \frac{[(2x-t)b(b-2x') + (2x'-t)(a-t)(a+t-2x)]t}{2(I'-I)}.$$

$$I, \text{ Axis } 1-1 = \frac{t(a-x)^3 + bx^3 - (b-t)(x-t)^3}{3}.$$

$$I', \text{ Axis } 2-2 = \frac{t(b-x')^3 + ax'^3 - (a-t)(x'-t)^3}{3}.$$

$$I'', \text{ Axis } 3-3 = \frac{I \cos^2 \alpha - I' \sin^2 \alpha}{\cos 2\alpha}.$$

axis AB is equal to its moment of inertia about the axis ab plus the product of its area by x^2. The moment of inertia for the standard merchant shapes of structural steel may be found from the tables given in this chapter. The distance d may be found from columns IX. and X., "Properties of Angles." This distance subtracted from D will give the distance x.

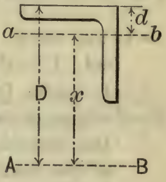


Fig. 1

The method of finding the moment of inertia for the most common combinations is indicated below. The column numbers refer to the columns in the table giving the properties of the section under consideration. A period between letters denotes multiplication.

Fig 2. *Moment of Inertia of Combination about Axis AB*

= twice the moment of inertia for beam a (col. II.) + that for beam b (col. III.).

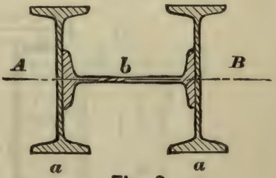


Fig. 2

Fig. 3. *Moment of Inertia of Combination about Axis CD*

= twice the area of beam a (col. I) $\times d^2$ + twice moment of inertia for beam a (col. III.) + that for beam b (col. II.).

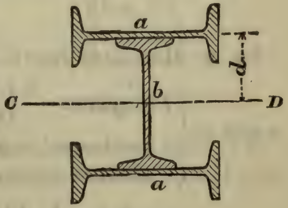


Fig. 3

Fig. 4. *Moment of Inertia of Combination about Axis AB*

= twice the moment of single channel in col. II.

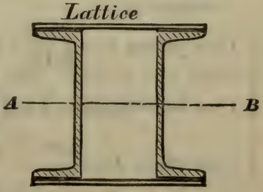


Fig. 4

Fig. 5. *Moment of Inertia of Combination about Axis CD*

= twice area of one channel (col. I.) $\times d^2$ + twice moment of inertia (col. III.).
d = distance of centre of gravity of the channel from centre line of the combination.

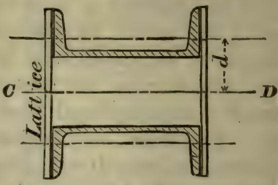


Fig. 5

Fig. 6. *Moment of Inertia of Combination about Axis AB*

= sum of $\begin{cases} I \text{ for plates} = 2\left(\dfrac{b.t^3}{12} + b.t.y^2\right); \\ I \text{ for channels} = \text{twice the moment in col. II.} \end{cases}$

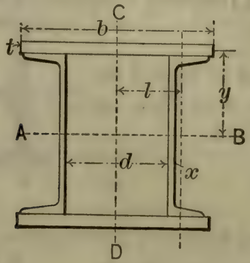


Fig. 6

Fig. 6. *Moment of Inertia about Axis CD*

= sum of $\begin{cases} I \text{ for plates} = \dfrac{2.t.b^3}{12}; \\ I \text{ for channels} = 2 \times (\text{area of one channel} \times l^2 + \text{moment of inertia, col. III.}). \end{cases}$

$l = \frac{1}{2}d + X$, col. IX.

Fig. 7. *Moment of Inertia of Combination about Axis AB* = sum of

$\begin{cases} I \text{ for plates } P = \left(\dfrac{b.t^3}{12} + b.t.x^2\right) \times 2; \\ I \text{ for four angles} = 4 \times \left(\begin{matrix}\text{moment of one} \\ \text{angle, col. III.}\end{matrix} + \begin{matrix}\text{area of one angle} \\ \text{times } y^2\end{matrix}\right); \\ I \text{ for plate } P_1 = \dfrac{t_1 d^3}{12}. \end{cases}$

$y = \frac{1}{2}d - l$, col. IX.

Fig. 7. *Moment of Inertia about Axis CD* $=$ sum of

$$\begin{cases} I \text{ for plates } P = \dfrac{2tb^3}{12}; \\ I \text{ for four angles} = 4 \times \begin{pmatrix} \text{moment of one} \\ \text{angle, col. II.} \end{pmatrix} + \begin{matrix} \text{area of one angle} \\ \text{times } l^2 \end{matrix} ; \\ I \text{ for plate } P_1 = \dfrac{d \cdot t_1^3}{12}. \end{cases}$$

$$l = d \text{ (col. X)} + \tfrac{1}{2}t_1.$$

Moment of inertia for Four Angles connected by Lattice and without Cover-plates.—Same as in middle line of above.

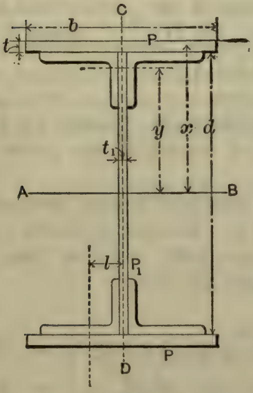


Fig. 7

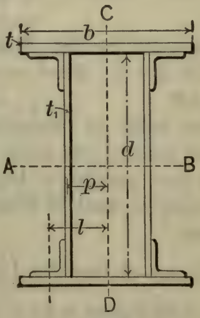


Fig. 8

Fig. 8. *Moment of Inertia of Combination about Axis AB.* Same as for Fig. 7, letting t_1 equal total thickness of web-plates.

Fig. 8. *Moment of Inertia about Axis CD* $=$ sum of

$$\begin{cases} I \text{ for flange-plates} = \dfrac{2tb^3}{12}; \\ I \text{ for four angles} = 4 \times \begin{pmatrix} \text{moment of one} \\ \text{angle, col. II.} \end{pmatrix} + \begin{matrix} \text{area of one angle} \\ \text{times } l^2 \end{matrix} ; \\ I \text{ for web-plates} = 2 \times \left(\dfrac{d \cdot t_1^3}{12} + d \cdot t_1 \cdot p^2 \right). \end{cases}$$

$$l = p + \tfrac{1}{2}t_1 + d \text{(col. X.)}.$$

Fig. 9. *Moment of Inertia of Two Angles.*

About axis AB : I = twice the moment of single angle, col. II.

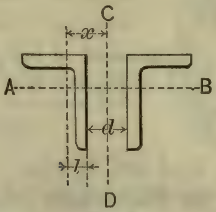


Fig. 9

About axis CD: $I=2\times\left(\begin{matrix}\text{moment of one}\\ \text{angle, col. III.}\end{matrix}+\begin{matrix}\text{area of one angle}\\ \text{times } x^2\end{matrix}\right)$;

$$x=\tfrac{1}{2}d+l \text{ (col. IX.)}.$$

EXAMPLE I.—Fig. 7, Let $b=12$; $t=\frac{1}{2}$. $d=30$; $t_1=\frac{1}{2}$. Angles $5\times3\frac{1}{2}\times\frac{1}{2}$. Find moment of inertia of the girder about axis AB.

From table of properties of standard angles, unequal legs, we find area of one angle to be 4. Moment of inertia of angle about axis parallel to long flange=4.05; distance from centre of gravity to back of long flange=.91.

Then

$$I \text{ for flange-plates}=2\times\left(\frac{12\times(\frac{1}{2})^3}{12}+12\times\tfrac{1}{2}\times(15\tfrac{1}{4})^2\right) = 2790.96$$

$$I \text{ for four angles } =4\times[4.05+4\times(15-.91)^2] = 3192.20$$

$$I \text{ for web-plate } =\frac{\frac{1}{2}\times30^3}{12} = 1125.00$$

$$I \text{ for whole section}=7108.16$$

It will be noticed that the moment of inertia of the flange-plates and angles about their own axes is so small, compared with the moment of the girder, that they might be omitted without any appreciable error.

In calculating the moment of inertia of riveted girders it is the custom with many engineers to let I = area of flange-plates and angles $\times\left(\frac{d}{2}\right)^2$, which in this case would give

$$I=28\times15^2=6300.$$

Example II.—Find the moment of inertia of two $6 \times 4 \times \frac{3}{8}$ nch angles, placed as in Fig. 9, d being made 1 inch.

Ans. Moment of inertia about axis $AB = 2 \times 13.47$ (see col. II., p. 303) $= 26.94$

For moment about axis CD we have

Area of one angle from col. I., p. 303, $= 3.61$; $I = 4.90$; $X = .5 + .94$ (col. IX.) $= 1.44$; then

I for both angles $= 2 \times [4.90 + 3.61 \times 1.44^2] = 24.76$.

Radius of Gyration of Compound Shapes.

A. *By Moment of Inertia.*

The radius of gyration of any combination is found by dividing the moment of inertia of the shape by the total metal area and taking the square root of the product.

Thus, the radius of gyration of the two angles in Example II.,

$$\text{about } AB = \sqrt{\frac{26.94}{7.22}} = 1.93;$$

$$\text{about } CD = \sqrt{\frac{24.76}{7.22}} = 1.85.$$

B. *Without Moment of Inertia.*

In the case of a pair of any shape without a web the value of the radius of gyration can always be readily found without considering the moment of inertia.

The radius of gyration for any section around an axis parallel to another axis passing through its centre of gravity is found as follows:

Let r = radius of gyration around axis through centre of gravity; R = radius of gyration around another axis parallel to above; d = distance between axes; then

$$R = \sqrt{d^2 + r^2}.$$

Thus, in Example II., the radius of gyration about the axis CD could have been obtained as follows: r = radius of one

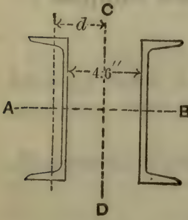
angle about its own axis parallel to $CD = 1.17$ (Column V.), $d = x$ (Fig. 9) $= .5 + .94$ (Column IX.) $= 1.44$.

$R = \sqrt{1.44^2 + 1.17^2} = 1.85$, the same result that we obtained by using the moment of inertia.

The radius about AB, Fig. 9, is the same as for one angle (Column II.).

When r is small compared with d, as is generally the case in latticed girders and columns, R may be taken as equal to d without material error.

EXAMPLE III.—Two 9-inch, 15-pound standard channel-bars are placed 4.6 inches apart, as in the figure; required the radius of gyration around axis CD for combined section.

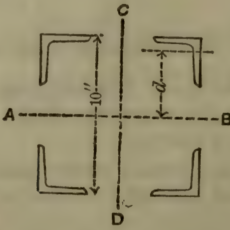


Ans. Find r, in Column V., p. 298 = 0.665; and $r^2 = .4422$.

Distance from base of channel to neutral axis, Column IX., is 0.59. One-half of $4.6 = 2.3 + .59 = 2.89$, the distance between neutral axis of single channel and of combined section; hence

$R = \sqrt{8.3521 + .4422} = 2.96$; or, for all practical purposes, $R = d$.

EXAMPLE IV.—Four $3 \times 3 \times \frac{1}{4}$-inch standard angles placed as shown form a column 10 inches square; find the radius of gyration.



Ans. From Column IV., p. 310, we find $r = 0.93$ and $r^2 = .8649$. The distance from base of angle to neutral axis, Column IX., is .84; hence, $d = 5 - .84 = 4.16$, or, $d^2 = 17.3056$, and

$$R = \sqrt{17.3056 + .8649} = 4.26.$$

Table I. will be found of considerable assistance when computing the moment of inertia of sections built with plates.

TABLE I.

MOMENTS OF INERTIA OF...... ......RECTANGLES.

Depth in inches.	Width of rectangle in inches.						
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$
2	.17	.21	.25	.29	.33	.38	.42
3	56	.70	.84	.98	1.13	1.27	1.41
4	1.33	1.67	2.00	2.33	2.67	3.00	3.33
5	2.60	3.26	3.91	4.56	5.21	5.86	6.51
6	4.50	5.63	6.75	7.88	9.00	10.13	11.25
7	7.15	8.93	10.72	12.51	14.29	16.08	17.86
8	10.67	13.33	16.00	18.67	21.33	24.00	26.67
9	15.19	18.98	22.78	26.58	30.38	34.17	37.97
10	20.83	26.04	31.25	36.46	41.67	46.87	52.08
11	27.73	34.66	41.59	48.53	55.46	62.39	69.32
12	36.00	45.00	54.00	63.00	72.00	81.00	90.00
13	45.77	57.21	68.66	80.10	91.54	102.98	114.43
14	57.17	71.46	85.75	100.04	114.33	128.63	142.92
15	70.31	87.89	105.47	123.05	140.63	158.20	175.78
16	85.33	106.67	128.00	149.33	170.67	192.00	213.33
17	102.35	127.94	153.53	179.12	204.71	230.30	255.89
18	121.50	151.88	182.25	212.63	243.00	273.38	303.75
19	142.90	178.62	214.34	250.07	285.79	321.52	357.24
20	166.67	208.33	250.00	291.67	333.33	375.00	416.67
21	192.94	241.17	289.41	337.64	385.88	434.11	482.34
22	221.83	277.29	332.75	388.21	443.67	499.13	554.58
23	253.48	316.85	380.22	443.59	506.96	570.33	633.70
24	288.00	360.00	432.00	504.00	576.00	648.00	720.00
25	325.52	406.90	488.28	569.66	651.04	732.42	813.80
26	366.17	457.71	549.25	640.79	732.33	823.88	915.42
27	410.06	512.58	615.09	717.61	820.13	922.64	1025.16
28	457.33	571.67	686.00	800.33	914.67	1029.00	1143.33
29	508.10	635.13	762.16	889.18	1016.21	1143.23	1270.26
30	562.50	703.13	843.75	984.38	1125.00	1265.63	1406.25
32	682.67	853.33	1024.00	1194.67	1365.33	1536.00	1706.67
34	818.83	1023.54	1228.25	1432.96	1637.67	1842.38	2047.08
36	972.00	1215.00	1458.00	1701.00	1944.00	2187.00	2430.00
38	1143.17	1428.96	1714.75	2000.54	2286.33	2572.13	2857.92
40	1333.33	1666.67	2000.00	2333.33	2666.67	3000.00	3333.33
42	1543.50	1929.38	2315.25	2701.13	3087.00	3472.88	3858.75
44	1774.67	2218.33	2662.00	3105.67	3549.33	3993.00	4436.67
46	2027.83	2534.79	3041.75	3548.71	4055.67	4562.63	5069.58
48	2304.00	2880.00	3456.00	4032.00	4608.00	5184.00	5760.00
50	2604.17	3255.21	3906.25	4557.29	5208.33	5859.38	6510.42
52	2929.33	3661.67	4394.00	5126.33	6591.00	5858.67	7323.33
54	3280.50	4100.63	4920.75	5740.88	6561.00	7381.13	8201.25
56	3658.67	4573.33	5488.00	6402.67	7317.33	8232.00	9146.67
58	4064.83	5081.04	6097.25	7113.46	8129.67	9145.87	10162.08
60	4500.00	5625.00	6750.00	7875.00	9000.00	10125.00	11250.00

TABLE I—*Continued.*

MOMENTS OF INERTIA OF...... ▮ ......RECTANGLES

Width of rectangle in inches. $\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	Depth in inches.
.46	.50	.54	.58	.63	.67	2
1.55	1.69	1.83	1.97	2.11	2.25	3
3.67	4.00	4.33	4.67	5.00	5.33	4
7.16	7.81	8.46	9.11	9.77	10.42	5
12.38	13.50	14.63	15.75	16.88	18.00	6
19.65	21.44	23.22	25.01	26.80	28.58	7
29.33	32.00	34.67	37.33	40.00	42.67	8
41.77	45.56	49.36	53.16	56.95	60.75	9
57.29	62.50	67.71	72.92	78.13	83.33	10
76.26	83.19	90.12	97.05	103.98	110.92	11
99.00	108.00	117.00	126.00	135.00	144.00	12
125.87	137.31	148.75	160.20	171.64	183.08	13
157.21	171.50	185.79	200.08	214.38	228.67	14
193.36	210.94	228.52	246.09	263.67	281.25	15
234.67	256.00	277.33	298.67	320.00	341.33	16
281.47	307.06	332.65	358.24	383.83	409.42	17
334.13	364.50	394.88	425.25	455.63	486.00	18
392.96	428.69	464.41	500.14	535.86	571.58	19
458.33	500.00	541.67	583.33	625.00	666.67	20
530.58	578.81	627.05	675.28	723.52	771.75	21
610.04	665.50	720.96	776.42	831.87	887.33	22
697.07	760.44	823.81	887.18	950.55	1013.92	23
792.00	864.00	936.00	1008.00	1080.00	1152.00	24
895.18	976.56	1057.94	1139.32	1220.70	1302.08	25
1006.96	1098.50	1190.04	1281.58	1373.13	1464.67	26
1127.67	1230.19	1332.70	1435.22	1537.73	1640.25	27
1257.67	1372.00	1486.33	1600.67	1715.00	1829.33	28
1397.29	1524.31	1651.34	1778.36	1905.39	2032.42	29
1546.88	1687.50	1828.13	1968.75	2109.38	2250.00	30
1877.33	2048.00	2218.67	2389.33	2560.00	2730.67	32
2251.79	2456.50	2661.21	2865.92	3070.63	3275.33	34
2673.00	2916.00	3159.00	3402.00	3645.00	3888.00	36
3143.71	3429.50	3715.29	4001.08	4286.88	4572.67	38
3666.67	4000.00	4333.33	4666.67	5000.00	5333.33	40
4244.63	4630.50	5016.38	5402.25	5788.13	6174.00	42
4880.33	5324.00	5767.67	6211.33	6655.00	7098.67	44
5576.54	6083.50	6590.46	7097.42	7604.38	8111.33	46
6336.00	6912.00	7488.00	8064.00	8640.00	9216.00	48
7161.46	7812.50	8463.54	9114.58	9765.63	10416.67	50
8055.67	8788.00	9520.33	10252.67	10985.00	11717.33	52
9021.38	9841.50	10661.63	11481.75	12301.88	13122.00	54
10061.33	10976.00	11890.67	12805.33	13720.00	14634.67	56
11178.29	12194.50	13210.71	14226.92	15243.12	16259.33	58
12375.00	13500.00	14625.00	15750.00	16875.00	18000.00	60

TABLE II.—RADII OF GYRATION FOR ROUND COLUMNS

Outside diameter of column in inches.	Thickness in inches varying by tenths.									
	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
	Corresponding radius of gyration in inches.									
2	.67	.64	.61	.58	.56	.54	.52	.51	.50	.50
3	1.03	.99	.96	.93	.90	.88	.85	.83	.81	.79
4	1.38	1.35	1.31	1.28	1.25	1.22	1.19	1.16	1.14	1.12
5	1.73	1.70	1.66	1.63	1.60	1.57	1.54	1.51	1.48	1.46
6	2.08	2.05	2.02	1.98	1.95	1.92	1.89	1.86	1.83	1.80
7	2.43	2.40	2.36	2.33	2.30	2.27	2.24	2.21	2.18	2.15
8	2.79	2.76	2.72	2.69	2.66	2.62	2.59	2.56	2.53	2.50
9	3.15	3.11	3.08	3.04	3.01	2.97	2.94	2.91	2.88	2.85
10	3.51	3.47	3.44	3.40	3.37	3.33	3.30	3.27	3.23	3.20
11	3.86	3.82	3.79	3.75	3.72	3.68	3.65	3.62	3.58	3.55
12	4.21	4.18	4.15	4.11	4.08	4.04	4.01	3.97	3.94	3.90

TABLE III.—RADII OF GYRATION FOR SQUARE COLUMNS.

Outside of column in inches.	Thickness in inches varying by tenths.									
	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0
	Corresponding radius of gyration in inches.									
2	.78	.74	.71	.68	.65	.63	.61	.59	.58	.58
3	1.18	1.14	1.11	1.08	1.04	1.01	.98	.96	.93	.91
4	1.59	1.55	1.51	1.47	1.44	1.41	1.38	1.35	1.32	1.29
5	2.00	1.96	1.92	1.89	1.85	1.81	1.78	1.75	1.71	1.68
6	2.41	2.37	2.33	2.29	2.25	2.21	2.18	2.15	2.11	2.08
7	2.82	2.78	2.74	2.70	2.66	2.62	2.58	2.55	2.51	2.48
8	3.23	3.19	3.15	3.11	3.07	3.03	2.99	2.96	2.92	2.89
9	3.63	3.59	3.55	3.51	3.48	3.44	3.40	3.36	3.32	3.29
10	4.04	4.00	3.96	3.92	3.88	3.84	3.80	3.77	3.73	3.70
11	4.45	4.41	4.37	4.33	4.29	4.25	4.21	4.17	4.13	4.10
12	4.86	4.82	4.78	4.74	4.70	4.66	4.62	4.58	4.54	4.51

Dimensions, Moments of Inertia, Radii of Gyration and Section Modulus of Standard Structural Shapes.

As in using steel in structural shapes one is practically confined to the choice of such shapes as are rolled by the mills, it is necessary to have at hand the dimensions and properties of those shapes to be able to calculate the necessary size to meet

special requirements for strength and the practical conditions of economy and framing. During the past fifteen years great changes have been made both in the material and shape of structural bars of steel and iron. At the present time the New Jersey Steel and Iron Company is the only manufacturer of wrought-iron beams in the country, to the writer's knowledge, all other mills rolling steel shapes, only, except perhaps small angles and bars.

The rolling mills which manufacture the most complete line of structural shapes are those of the Carnegie Steel Co., Cambria Iron Co., Jones & Laughlins, Passaic Rolling Mill Co., Pencoyd Iron Works, and the Phœnix Iron Co. In general, the products of these mills agree in shape quite closely, especially for beams and channels. This is particularly true of the shapes rolled by the first three of the companies named above.

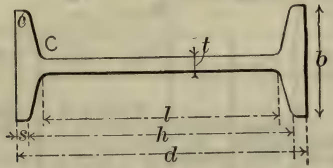
The standard steel beams and channels given in the following pages are rolled by all six of the mills, with the exception of the 24-inch beams which are not rolled by the Passaic and Phœnix mills. Some of the mills also roll additional weights. Thus the Pencoyd Iron Works rolls 18-inch beams up to 90 lbs., and 6-inch beams up to 46 lbs. per foot. Except for the 18-inch beams, only the properties of the standard sizes are given in this book.

The following tables of properties of structural shapes have been compiled from the 1900 publication of the Carnegie Steel Company, except in the case of shapes not rolled by them. It may be well to state that the tables of properties for the various structural shapes, published by the companies named above, do not agree exactly, even for the same weights, but the differences are not of practical importance. The tables of the Cambria Iron Company and of the Carnegie Steel Company agree the closest, for beams and channels. As angles are very extensively used for a great many purposes, the properties for all sizes rolled are given, and also a table showing from which mills the different sizes may be obtained. Naturally it will generally be advantageous to use a size that is rolled by several mills.

The properties for grooved steel, given on p. 300, were computed by the author from the dimensions given by the manufacturers. These small channels are quite extensively used in connection with suspended ceilings, and other fireproof constructions, and it is believed that the table will be found useful by many. The Tables A, B, C, and D will be found very con-

venient when computing the strength of struts formed of a pair of channels and angles.

Standard Steel Beams and Channels.

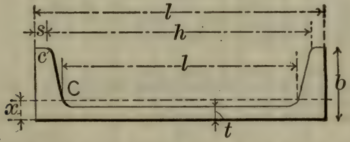


The following data are common to all standard **I** beams and channels, with the exceptions stated:

$$c = \tfrac{6}{10} \text{ minimum web;}$$

$$C = \text{minimum web} + \tfrac{1}{10} \text{ inch.}$$

s = thickness of web = t, minimum for all beams except 20″ I's and 24″ I's.



For 20″ beam $s = .55''$, $t = .50''$.

For 24″ beams $s = .60''$, $t = .50''$.

For 20″ beam, 80 lbs., $s = .65''$, $t = .60''$.

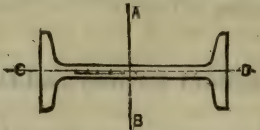
The slope of flange of all beams and channels is $16\frac{2}{3}$ per cent.

$= 9° - 27' - 44'' = 2''$ per foot.

Weight per foot = area × 3.4.

When ordering I beams, channels, or angles, the weight or thickness should be given, but not both.

PROPERTIES OF STANDARD STEEL I BEAMS.



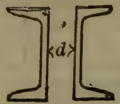
Depth of beam.	W'ght per foot, lbs.	I. Area, sq. in.	Thickness of web, in.	Width of flange, in.	II. Moment of inertia. Axis AB. I.	III. Moment of inertia. Axis CD. I'.	IV. Radius of gyration. Axis AB. r.	V. Radius of gyration. Axis CD. r'.	VII. Section modulus. Axis AB. R.
24	80.00	23.32	0.500	7.000	2087.9	42.86	9.46	1.36	174.0
	85.00	25.00	0.570	7.070	2168.6	44.35	9.31	1.33	180.7
	90.00	26.47	0.631	7.131	2239.1	45.70	9.20	1.31	186.6
	95.00	27.94	0.692	7.192	2309.6	47.10	9.09	1.30	192.5
	100.00	29.41	0.754	7.254	2380.3	48.56	9.00	1.28	198.4
20	65.00	19.08	0.500	6.250	1169.6	27.86	7.83	1.21	117.0
	70.00	20.59	0.575	6.325	1219.9	29.04	7.70	1.19	122.0
	75.00	22.06	0.649	6.399	1268.9	30.25	7.58	1.17	126.9
20	80.00	23.73	0.600	7.000	1466.5	45.81	7.86	1.39	146.7
	85.00	25.00	0.663	7.063	1508.7	47.25	7.77	1.37	150.9
	90.00	26.47	0.737	7.137	1557.8	48.98	7.67	1.36	155.8
	95.00	27.94	0.810	7.210	1606.8	50.78	7.58	1.35	160.7
	100.00	29.41	0.884	7.284	1655.8	52.65	7.50	1.34	165.6
18	55.00	15.93	0.460	6.000	795.6	21.19	7.07	1.15	88.4
	60.00	17.65	0.555	6.095	841.8	22.38	6.91	1.13	93.5
	65.00	19.12	0.637	6.177	881.5	23.47	6.79	1.11	97.9
	70.00	20.59	0.719	6.259	921.3	24.62	6.69	1.09	102.4
18*	75.00	22.05	0.71	6.58	1023.5	31.67	6.81	1.20	113.7
	80.00	23.53	0.79	6.66	1063.4	33.12	6.72	1.19	118.2
	85.00	25.00	0.74	7.00	1149.6	44.18	6.78	1.33	127.7
	90.00	26.46	0.82	7.08	1188.0	46.03	6.70	1.32	132.0
15	42.00	12.48	0.410	5.500	441.7	14.62	5.95	1.08	58.9
	45.00	13.24	0.460	5.550	455.8	15.09	5.87	1.07	60.8
	50.00	14.71	0.558	5.648	483.4	16.04	5.73	1.04	64.5
	55.00	16.18	0.656	5.746	511.0	17.06	5.62	1.02	68.1
15	60.00	17.67	0.590	6.000	609.0	25.96	5.87	1.21	81.2
	65.00	19.12	0.686	6.096	636.0	27.42	5.77	1.20	84.8
	70.00	20.59	0.784	6.194	663.6	29.00	5.68	1.19	88.5
	75.00	22.06	0.882	6.292	691.2	30.68	5.60	1.18	92.2
15	80.00	23.81	0.810	6.400	795.5	41.76	5.78	1.32	106.1
	85.00	25.00	0.889	6.479	817.8	43.57	5.72	1.32	109.0
	90.00	26.47	0.987	6.577	845.4	45.91	5.65	1.32	112.7
	95.00	27.94	1.085	6.675	872.9	48.37	5.59	1.32	116.4
	100.00	29.41	1.184	6.774	900.5	50.98	5.53	1.31	120.1
12	31.50	9.26	0.350	5.000	215.8	9.50	4.83	1.01	36.0
	35.00	10.29	0.436	5.086	228.3	10.07	4.71	0.99	38.0

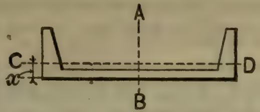
* Rolled only by Pencoyd and Passaic Mills.

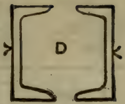
PROPERTIES OF STANDARD STEEL I BEAMS.
(Continued.)

		I.			II.	III.	IV.	V.	VII.
Depth of beam.	W'ght per foot, lbs.	Area, sq. in.	Thickness of web, in.	Width of flange, in.	Moment of inertia. Axis AB. *I.*	Moment of inertia. Axis CD. *I'.*	Radius of gyration. Axis AB. *r.*	Radius of gyration. Axis CD. *r'.*	Section modulus, Axis AB. *R.*
12	40.00	11.84	0.460	5.250	268.9	13.81	4.77	1.08	44.8
	45.00	13.24	0.576	5.366	285.7	14.89	4.65	1.06	47.6
	50.00	14.71	0.699	5.489	303.3	16.12	4.54	1.05	50.6
	55.00	16.18	0.822	5.612	321.0	17.46	4.45	1.04	53.5
10	25.00	7.37	0.310	4.660	122.1	6.89	4.07	0.97	24.4
	30.00	8.82	0.455	4.805	134.2	7.65	3.90	0.93	26.8
	35.00	10.29	0.602	4.952	146.4	8.52	3.77	0.91	29.3
	40.00	11.76	0.749	5.099	158.7	9.50	3.67	0.90	31.7
9	21.00	6.31	0.290	4.330	84.9	5.16	3.67	0.90	18.9
	25.00	7.35	0.406	4.446	91.9	5.65	3.54	0.88	20.4
	30.00	8.82	0.569	4.609	101.9	6.42	3.40	0.85	22.6
	35.00	10.29	0.732	4.772	111.8	7.31	3.29	0.84	24.8
8	18.00	5.33	0.270	4.000	56.9	3.78	3.27	0.84	14.2
	20.50	6.03	0.357	4.087	60.6	4.07	3.17	0.82	15.1
	23.00	6.76	0.449	4.179	64.5	4.39	3.09	0.81	16.1
	25.50	7.50	0.541	4.271	68.4	4.75	3.02	0.80	17.1
7	15.00	4.42	0.250	3.660	36.2	2.67	2.86	0.78	10.4
	17.50	5.15	0.353	3.763	39.2	2.94	2.76	0.76	11.2
	20.00	5.88	0.458	3.868	42.2	3.24	2.68	0.74	12.1
6	12.25	3.61	0.230	3.330	21.8	1.85	2.46	0.72	7.3
	14.75	4.34	0.352	3.452	24.0	2.09	2.35	0.69	8.0
	17.25	5.07	0.475	3.575	26.2	2.36	2.27	0.68	8.7
5	9.75	2.87	0.210	3.000	12.1	1.23	2.05	0.65	4.8
	12.25	3.60	0.357	3.147	13.6	1.45	1.94	0.63	5.4
	14.75	4.34	0.504	3.294	15.2	1.70	1.87	0.63	6.1
4	7.50	2.21	0.190	2.660	6.0	0.77	1.64	0.59	3.0
	8.50	2.50	0.263	2.733	6.4	0.85	1.59	0.58	3.2
	9.50	2.79	0.337	2.807	6.7	0.93	1.55	0.58	3.4
	10.50	3.09	0.410	2.880	7.1	1.01	1.52	0.57	3.6
3	5.50	1.63	0.170	2.330	2.5	0.46	1.23	0.53	1.7
	6.50	1.91	0.263	2.423	2.7	0.53	1.19	0.52	1.8
	7.50	2.21	0.361	2.521	2.9	0.60	1.15	0.52	1.9

PROPERTIES OF STANDARD STEEL CHANNELS.





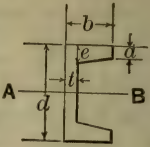


Depth of channel.	Weight per foot.	I. Area.	Thickness of web.	Width of flange.	II. Moment of inertia. Axis AB. I.	III. Moment of inertia. Axis CD. I'.	IV. Radius of gyration. Axis AB. r.	V. Radius of gyration. Axis CD. r'.	VII. Section modulus. Axis AB. R.	IX. Distance of centre of gravity from back. X.	d.	D.
15	33.00	9.90	0.400	3.400	312.6	8.23	5.62	.912	41.7	0.794	9.50	12.82
	35.00	10.29	0.426	3.426	320.0	8.48	5.58	.908	42.7	0.789	9.43	12.73
	40.00	11.76	0.524	3.524	347.5	9.39	5.43	.893	46.3	0.783	9.15	12.42
	45.00	13.24	0.622	3.622	375.1	10.29	5.32	.882	50.0	0.788	8.92	12 23
	50.00	14.71	0.720	3.720	402.7	11.22	5.23	.873	53.7	0.803	8.71	12.06
	55.00	16.18	0.818	3.818	430.2	12.19	5.16	.868	57.4	0.823	8.53	11.96
12	20.50	6.03	0.280	2.940	128.1	3.91	4.61	.805	21.4	0.704	7.67	10.63
	25.00	7.35	0.390	3.050	144.0	4.53	4.43	.785	24.0	0.678	7.36	10.21
	30.00	8.82	0.513	3.173	161.7	5.21	4.28	.768	26.9	0.677	7.07	9.91
	35.00	10.29	0.636	3.296	179.3	5.90	4.17	.757	29.9	0.694	6.81	9.73
	40.00	11.76	0.758	3.418	197.0	6.63	4.09	.751	32.8	0.722	6.60	9.62
10	15.00	4.46	0.240	2.600	66.9	2.30	3.87	.718	13.4	0.639	6.33	9.02
	20.00	5.88	0.382	2.742	78.7	2.85	3.66	.696	15.7	0.609	5.97	8.54
	25.00	7.35	0.529	2.889	91.0	3.40	3.52	.680	18.2	0.620	5.67	8.28
	30.00	8.82	0.676	3.036	103.2	3.99	3.42	.672	20.6	0.651	5.40	8.14
	35.00	10.29	0.823	3.183	115.5	4.66	3.35	.672	23.1	0.695	5.17	8.09
9	13.25	3.89	0.230	2.430	47.3	1.77	3.49	.674	10.5	0.607	5.63	8.19
	15.00	4.41	0.288	2.488	50.9	1.95	3.40	.665	11.3	0.590	5.49	7.98
	20.00	5.88	0.452	2.652	60.8	2.45	3.21	.646	13.5	0.585	5.12	7.59
	25.00	7.35	0.615	2.815	70.7	2.98	3.10	.637	15.7	0.615	4.84	7.43

PROPERTIES OF STANDARD STEEL CHANNELS—(*Continued*).

		I.			II.	III.	IV.	V.	VII.	IX.		
					Moment of inertia.		Radius of gyration.					
Depth of channel.	Weight per foot.	Area.	Thickness of web.	Width of flange.	Axis AB. *I*.	xis CD. *I′*.	Axis AB. *r*.	Axis CD. *r′*.	Section modulus. Axis AB. *R*.	Distance of centre of gravity from back. *X*.	*d*.	*D*.
8	11.25	3.35	0.220	2.260	32.3	1.33	3.11	.630	8.1	0.576	4.94	7.37
	13.75	4.04	0.307	2.347	36.0	1.55	2.98	.619	9.0	0.557	4.72	7.07
	16.25	4.78	0.399	2.439	39.9	1.78	2.89	.610	10.0	0.556	4.54	6.89
	18.75	5.51	0.490	2.530	43.8	2.01	2.82	.603	11.0	0.567	4.38	6.77
	21.25	6.25	0.582	2.622	47.8	2.25	2.77	.600	11.9	0.587	4.23	6.71
7	9.75	2.85	0.210	2.090	21.1	0.98	2.72	.586	6.0	0.546	4.22	6.53
	12.25	3.60	0.318	2.198	24.2	1.19	2.59	.575	6.9	0.528	3.99	6.21
	14.75	4.34	0.423	2.303	27.2	1.40	2.50	.568	7.8	0.535	3.80	6.07
	17.25	5.07	0.528	2.408	30.2	1.62	2.44	.564	8.6	0.555	3.64	5.99
	19.75	5.81	0.633	2.513	33.2	1.85	2.39	.565	9.5	0.583	3.48	5.94
6	8.00	2.38	0.200	1.920	13.0	0.70	2.34	.542	4.3	0.517	3.52	5.71
	10.50	3.09	0.318	2.038	15.1	0.88	2.21	.534	5.0	0.503	3.28	5.42
	13.00	3.82	0.440	2.160	17.3	1.07	2.13	.529	5.8	0.517	3.09	5.29
	15.50	4.56	0.563	2.283	19.5	1.28	2.07	.529	6.5	0.546	2.91	5.23
5	6.50	1.95	0.190	1.750	7.4	0.48	1.95	.498	3.0	0.489	2.79	4.87
	9.00	2.65	0.330	1.890	8.9	0.64	1.83	.493	3.5	0.481	2.56	4.62
	11.50	3.38	0.477	2.037	10.4	0.82	1.75	.493	4.2	0.508	2.34	4.51
4	5.25	1.55	0.180	1.580	3.8	0.32	1.56	.453	1.9	0.464	2.06	4.04
	6.25	1.84	0.252	1.652	4.2	0.38	1.51	.454	2.1	0.458	1.96	3.93
	7.25	2.13	0.325	1.725	4.6	0.44	1.46	.455	2.3	0.463	1.85	3.84
3	4.00	1.19	0.170	1.410	1.6	0.20	1.17	.409	1.1	0.443	1.31	3.22
	5.00	1.47	0.264	1.504	1.8	0.25	1.12	.415	1.2	0.443	1.19	3.12
	6.00	1.76	0.362	1.602	2.1	0.31	1.08	.421	1.4	0.459	1.07	3.07

DIMENSIONS AND PROPERTIES OF GROOVED STEEL, OR SMALL CHANNELS.



No. of Sec.	*d*	*b*	*t*	*e*	*a*	W'ght per foot.	Area.	Moment of inertia.[1]	Section modulus.[1]	Coefficient of str'th.[2]
	ins.	ins.	ins.	ins.	ins.	lbs.	sq. ins.			lbs.
1	*2¼	1.37	.25	.25	.25	3.80	1.12	0.80	0.71	7570
2	*2	1.09	.22	.31	.18	2.90	0.87	.48	.48	5120
3	*2	1.18	.31	.....	.....	3.60	1.06	.54	.54	5760
4	†2	1.25	.25	.31	.25	3.60	1.062	.585	.585	6240
5	‡2	1.00	3/16	.25	.22	2.6	0.756	.423	.423	4512
6	‡2	⅝	.25	.19	⅛	2.0	0.619	.2665	.266	2836
7	*1¾	0.59	.09	.25	.09	1.13	0.33	.15	.17	1815
8	†1½	0.75	⅛	⅛	⅛	1.32	0.344	.1083	.144	1536
9	‡1½	⅝	.19	.2	.14	1.46	0.433	.1194	.159	1736
10	‡1¼	0.5	⅛	3/16	⅛	0.94	0.273	.0557	.089	950
11	‡1⅛	0.56	.19	.19	⅛	1.12	0.330	.0500	.088	939
12	†1⅛	⅝	⅛	⅛	⅛	1.00	0.266	.0462	.082	874
13	‡1	0.5	⅛	3/16	⅛	0.83	0.242	.0315	.063	672
14	‡1	0.39	⅛	3/16	⅛	0.68	0.208	.0253	.050	532
15	‡⅞	7/16	⅛	.16	.11	0.67	0.183	.0185	.042	448
16	‡⅞	0.42	.11	3/16	⅛	0.69	0.193	.0189	.043	458
17	‡¾	⅜	⅛	.16	.09	0.53	0.156	.0095	.025	266

[1] Axis AB.
[2] Computed for fibre stress of 16,000 lbs. per square inch.
* Rolled by the Pencoyd Iron Works.
† " " " Illinois Steel Company.
‡ " " " Jones & Laughlins, Ltd.

DIMENSIONS OF CAR TRUCK CHANNELS.

(Rolled by Carnegie Steel Co. and Jones & Laughlins, Ltd.)

d	*b*	*t*	*e*	*a*	Weight.	Area.
13	4	0.375	0.88	0.34	32	9.41
12	2.64	0.31	0.75	0.34	21.33	6.27

ANGLES.

The following table has been compiled to show all the various sizes of angles that are rolled, and also by what companies. The abbreviations indicate the companies that roll that particular size. The word *all* shows that the size is rolled by all of the five companies included in the list. The abbreviations refer

to the following companies: Cam., Cambria Iron Co.; Car., The Carnegie Steel Co.; J. & L., Jones & Laughlins; Pas., Passaic Rolling Mill Co.; Pen., Pencoyd Iron Works.

ANGLES WITH UNEQUAL LEGS.

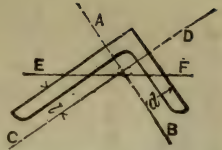
Size.	
8×6	Pen.
$7 \times 3\frac{1}{2}$	Car., Pen.
$6\frac{1}{2} \times 4$	Pen.
6×4	All.
$6 \times 3\frac{1}{2}$	Cam., Car., J. & L., Pen.
$5\frac{5}{8} \times 5$	Cam.
$5\frac{1}{2} \times 3\frac{1}{2}$	Pen.
5×4	Cam., Car., J. & L., Pen.
$5 \times 3\frac{1}{2}$	All.
5×3	All.
$4\frac{1}{2} \times 3$	Cam., Car., Pas., Pen.
$4 \times 3\frac{1}{2}$	All.
4×3	All.
$3\frac{3}{4} \times 2\frac{5}{8}$	J. & L.
$3\frac{1}{2} \times 3$	All.
$3\frac{1}{2} \times 2\frac{1}{2}$	All.
$3\frac{1}{2} \times 2$	Pen.
$3\frac{1}{4} \times 2$	Car., J. & L.
$3 \times 2\frac{1}{2}$	All.
3×2	All.
$2\frac{1}{2} \times 2$	All.
$2\frac{1}{2} \times 1\frac{3}{4}$	Cam.
$2\frac{1}{2} \times 1\frac{1}{2}$	Cam.
$2\frac{1}{2} \times 1\frac{1}{4}$	Cam.
$2\frac{1}{4} \times 1\frac{1}{2}$	Car., Pas., Pen.
$2 \times 1\frac{3}{4}$	Pas.
$2 \times 1\frac{1}{2}$	Cam., Pen.
$2 \times 1\frac{3}{8}$	Cam., Car.
$2 \times 1\frac{1}{4}$	Pen.
$1\frac{3}{8} \times 1\frac{1}{8}$	Pas.
$1\frac{3}{8} \times 1$	Car.
$1\frac{3}{8} \times \frac{7}{8}$	J. & L.
$1 \times \frac{5}{8}$	J. & L.

ANGLES WITH EQUAL LEGS.

Size.	
8×8	Car., Pen.
6×6	All.
5×5	All.
$4\frac{1}{2} \times 4\frac{1}{2}$	Cam.
4×4	All.
$3\frac{1}{2} \times 3\frac{1}{2}$	All.
$3\frac{1}{4} \times 3\frac{1}{4}$	J. & L.
3×3	All.
$2\frac{3}{4} \times 2\frac{3}{4}$	Cam., Car., J. & L., Pen.
$2\frac{1}{2} \times 2\frac{1}{2}$	All.
$2\frac{1}{4} \times 2\frac{1}{4}$	All.
2×2	All.
$1\frac{3}{4} \times 1\frac{3}{4}$	All.
$1\frac{1}{2} \times 1\frac{1}{2}$	All.
$1\frac{1}{4} \times 1\frac{1}{4}$	All.
1×1	All.
$\frac{7}{8} \times \frac{7}{8}$	Car., Pas.
$\frac{3}{4} \times \frac{3}{4}$	Cam., Car., J. & L., Pas.

PROPERTIES OF STANDARD AND SPECIAL ANGLES.

ANGLES WITH UNEQUAL LEGS.



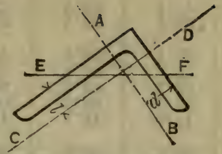
			I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.
Size, inches.	Thickness of metal, inches.	Weight per foot, pounds.	Area of section, sq. ins.	Moments of inertia, *I*.		Radii of gyration, *r*.			Section modulus, *R*.		Perpendicular distances from centre of gravity to back of flanges.	
				Axis AB.	Axis CD.	Axis AB.	Axis CD.	Axis EF.	Axis AB.	Axis CD.	To back of longer flange, *l*.	To back of shorter flange, *d*.
8¼ × 6¼	1	45.6	13.41	85.34	41.67	2.53	1.77	1.37	15.43	9.20	1.72	2.72
8⅛ × 6⅛	3/4	33.8	9.94	64.83	31.70	2.54	1.78	1.35	11.73	7.00	1.59	2.59
8 × 6	1/2	23.0	6.76	44.33	21.73	2.56	1.79	1.34	8.03	4.80	1.47	2.47
	1	32.3	9.50	45.37	7.53	2.19	0.89	0.88	10.58	2.96	0.96	2.71
	15/16	30.5	8.97	43.13	7.18	2.19	0.89	0.88	10.00	2.80	0.94	2.69
	7/8	28.7	8.42	40.82	6.83	2.20	0.90	0.88	9.42	2.64	0.92	2.67
	13/16	26.8	7.87	38.45	6.46	2.21	0.91	0.88	8.82	2.48	0.89	2.64
	3/4	24.9	7.31	35.99	6.08	2.22	0.91	0.88	8.22	2.31	0.87	2.62
7 × 3½	11/16	23.0	6.75	33.47	5.69	2.23	0.92	0.89	7.60	2.14	0.85	2.60
	5/8	21.0	6.17	30.86	5.28	2.24	0.93	0.89	6.97	1.97	0.82	2.57
	9/16	19.0	5.59	28.18	4.86	2.25	0.93	0.89	6.33	1.80	0.80	2.55
	1/2	17.0	5.00	25.41	4.41	2.25	0.94	0.89	5.68	1.62	0.78	2.53
	7/16	15.0	4.40	22.56	3.95	2.26	0.95	0.89	5.01	1.47	0.75	2.50
6⅞ × 4⅜	15/16	31.9	9.48	42.40	12.91	2.12	1.17	0.99	9.58	4.07	1.20	2.45
6½ × 4	3/8	12.9	3.80	16.83	5.03	2.10	1.15	0.93	3.87	1.62	0.90	2.15

		Weight.	Area.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.
6×4	1	30.6	9.00	30.75	10.75	1.85	1.09	0.85	8.02	3.79	1.17	2.17
	15/16	28.9	8.50	29.26	10.26	1.86	1.10	0.85	7.59	3.59	1.14	2.14
	7/8	27.2	7.99	27.73	9.75	1.86	1.11	0.86	7.15	3.39	1.12	2.12
	13/16	25.4	7.47	26.15	9.23	1.87	1.11	0.86	6.70	3.18	1.10	2.10
	3/4	23.6	6.94	24.51	8.68	1.88	1.12	0.86	6.25	2.97	1.08	2.08
	11/16	21.8	6.41	22.82	8.11	1.89	1.13	0.86	5.78	2.76	1.06	2.06
	5/8	20.0	5.86	21.07	7.52	1.90	1.13	0.86	5.31	2.54	1.03	2.03
	9/16	18.1	5.31	19.26	6.91	1.90	1.14	0.87	4.83	2.31	1.01	2.01
	1/2	16.2	4.75	17.40	6.27	1.91	1.15	0.87	4.33	2.08	0.99	1.99
	7/16	14.3	4.18	15.46	5.60	1.92	1.16	0.87	3.83	1.85	0.96	1.96
	3/8	12.3	3.61	13.47	4.90	1.93	1.17	0.88	3.32	1.60	0.94	1.94
6×3½	1	28.9	8.50	29.24	7.21	1.85	0.92	0.74	7.83	2.90	1.01	2.26
	15/16	27.3	8.03	27.84	6.88	1.86	0.93	0.74	7.41	2.74	0.99	2.24
	7/8	25.7	7.55	26.38	6.55	1.87	0.93	0.75	6.98	2.59	0.97	2.22
	13/16	24.0	7.06	24.89	6.20	1.88	0.94	0.75	6.55	2.43	0.95	2.20
	3/4	22.3	6.56	23.34	5.84	1.89	0.94	0.75	6.10	2.27	0.93	2.18
	11/16	20.6	6.06	21.74	5.47	1.89	0.95	0.75	5.65	2.11	0.90	2.15
	5/8	18.9	5.55	20.08	5.08	1.90	0.96	0.75	5.19	1.94	0.88	2.13
	9/16	17.1	5.03	18.37	4.67	1.91	0.96	0.75	4.72	1.77	0.86	2.11
	1/2	15.3	4.50	16.59	4.25	1.92	0.97	0.76	4.24	1.59	0.83	2.08
	7/16	13.5	3.97	14.76	3.81	1.93	0.98	0.76	3.75	1.41	0.81	2.06
	3/8	11.7	3.42	12.86	3.34	1.94	0.99	0.77	3.25	1.23	0.79	2.04
5⅝×5	11/16	23.2	6.83	20.54	15.24	1.73	1.49	1.03	5.29	4.27	1.43	1.74
	7/16	15.2	4.46	13.94	10.39	1.77	1.53	1.04	3.51	2.84	1.34	1.65
5¾×3¾	5/8	17.9	5.47	17.62	5.85	1.79	1.03	0.84	4.66	2.10	0.97	1.97
5½×3½	3/8	11.0	3.23	10.15	3.28	1.77	1.01	0.81	2.76	1.22	0.82	1.82
5×4	7/8	24.2	7.11	16.42	9.23	1.52	1.14	0.84	4.99	3.31	1.21	1.71
	3/4	21.1	6.19	14.60	8.23	1.54	1.15	0.84	4.37	2.90	1.16	1.66
	5/8	17.8	5.23	12.61	7.14	1.55	1.17	0.84	3.73	2.48	1.12	1.62
	1/2	14.5	4.25	10.46	5.96	1.57	1.18	0.85	3.05	2.04	1.07	1.57
	3/8	11.0	3.23	8.14	4.67	1.59	1.20	0.86	2.34	1.57	1.03	1.53
5×3½	7/8	22.7	6.67	15.67	6.21	1.53	0.96	0.75	4.88	2.52	1.04	1.79
	13/16	21.3	6.25	14.81	5.89	1.54	0.97	0.75	4.58	2.37	1.02	1.77

PROPERTIES OF STANDARD AND SPECIAL ANGLES.

ANGLES WITH UNEQUAL LEGS.

(Continued.)



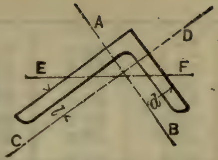
			I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.
Size, inches.	Thickness of metal, inches.	Weight per foot, pounds.	Area of section, sq. ins.	Moments of inertia. *I.*		Radii of gyration. *r.*			Section modulus. *R.*		Perpendicular distances from centre of gravity to back of flanges.	
				Axis AB.	Axis CD.	Axis AB.	Axis CD.	Axis EF.	Axis AB.	Axis CD.	To back of longer flange, *l*.	To back of shorter flange, *d*.
5×3½	3/4	19.8	5.81	13.92	5.55	1.55	0.98	0.75	4.28	2.22	1.00	1.75
	11/16	18.3	5.37	12.99	5.20	1.56	0.98	0.75	3.97	2.06	0.97	1.72
	5/8	16.8	4.92	12.03	4.83	1.56	0.99	0.75	3.65	1.90	0.95	1.70
	9/16	15.2	4.47	11.03	4.45	1.57	1.00	0.75	3.32	1.73	0.93	1.68
	1/2	13.6	4.00	9.99	4.05	1.58	1.01	0.75	2.99	1.56	0.91	1.66
	7/16	12.0	3.53	8.90	3.63	1.59	1.01	0.76	2.64	1.39	0.88	1.63
	3/8	10.4	3.05	7.78	3.18	1.60	1.02	0.76	2.29	1.21	0.86	1.61
	5/16	8.7	2.56	6.60	2.72	1.61	1.03	0.76	1.94	1.02	0.84	1.59
5×3	13/16	19.9	5.84	13.98	3.71	1.55	0.80	0.64	4.45	1.74	0.86	1.86
	3/4	18.5	5.44	13.15	3.51	1.55	0.80	0.64	4.16	1.63	0.84	1.84
	11/16	17.1	5.03	12.28	3.29	1.56	0.81	0.64	3.86	1.51	0.82	1.82
	5/8	15.7	4.61	11.37	3.06	1.57	0.82	0.64	3.55	1.39	0.80	1.80
	9/16	14.2	4.18	10.43	2.83	1.58	0.82	0.65	3.23	1.27	0.77	1.77
	1/2	12.8	3.75	9.45	2.58	1.59	0.83	0.65	2.91	1.15	0.75	1.75
	7/16	11.3	3.31	8.43	2.32	1.60	0.84	0.65	2.58	1.02	0.73	1.73
	3/8	9.8	2.86	7.37	2.04	1.61	0.84	0.65	2.24	0.89	0.70	1.70
	5/16	8.2	2.40	6.26	1.75	1.61	0.85	0.66	1.89	0.75	0.68	1.68

		Weight.	Area.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.
4½×3	13/16	18.5	5.43	10.33	3.60	1.38	0.81	0.64	3.62	1.71	0.90	1.65
	3/4	17.2	5.06	9.73	3.40	1.39	0.82	0.64	3.38	1.60	0.88	1.63
	11/16	15.9	4.68	9.10	3.19	1.39	0.83	0.64	3.14	1.49	0.85	1.60
	5/8	14.6	4.30	8.44	2.98	1.40	0.83	0.64	2.89	1.37	0.83	1.58
	9/16	13.3	3.90	7.75	2.75	1.41	0.85	0.64	2.64	1.25	0.81	1.56
	1/2	11.9	3.50	7.04	2.51	1.42	0.85	0.65	2.37	1.13	0.79	1.54
	7/16	10.5	3.09	6.29	2.25	1.43	0.85	0.65	2.10	1.01	0.76	1.51
	3/8	9.1	2.67	5.50	1.98	1.44	0.86	0.66	1.83	0.88	0.74	1.49
	5/16	7.7	2.25	4.69	1.73	1.44	0.86	0.66	1.54	0.76	0.72	1.47
4×3½	13/16	18.5	5.43	7.77	5.49	1.19	1.01	0.72	2.92	2.30	1.11	1.36
	3/4	17.2	5.06	7.32	5.18	1.20	1.01	0.72	2.75	2.15	1.09	1.34
	11/16	15.9	4.68	6.86	4.86	1.21	1.02	0.72	2.56	2.00	1.07	1.32
	5/8	14.6	4.30	6.37	4.52	1.22	1.03	0.72	2.35	1.84	1.04	1.29
	9/16	13.3	3.90	5.86	4.17	1.23	1.03	0.72	2.15	1.68	1.02	1.27
	1/2	11.9	3.50	5.32	3.79	1.23	1.04	0.72	1.93	1.52	1.00	1.25
	7/16	10.5	3.09	4.76	3.40	1.24	1.05	0.72	1.72	1.35	0.98	1.23
	3/8	9.1	2.67	4.18	2.99	1.25	1.06	0.73	1.50	1.18	0.96	1.21
	5/16	7.7	2.25	3.56	2.59	1.26	1.07	0.73	1.26	1.01	0.93	1.18
4×3	13/16	17.1	5.03	7.34	3.47	1.21	0.83	0.64	2.87	1.68	0.94	1.44
	3/4	16.0	4.69	6.93	3.28	1.22	0.84	0.64	2.68	1.57	0.92	1.42
	11/16	14.8	4.34	6.49	3.08	1.22	0.84	0.64	2.49	1.46	0.89	1.39
	5/8	13.6	3.98	6.03	2.87	1.23	0.85	0.64	2.30	1.35	0.87	1.37
	9/16	12.3	3.62	5.55	2.66	1.24	0.86	0.64	2.09	1.23	0.85	1.35
	1/2	11.1	3.25	5.05	2.42	1.25	0.86	0.64	1.89	1.12	0.83	1.33
	7/16	9.8	2.87	4.52	2.18	1.25	0.87	0.64	1.68	0.99	0.80	1.30
	3/8	8.5	2.48	3.96	1.92	1.26	0.88	0.64	1.46	0.87	0.78	1.28
	5/16	7.1	2.09	3.38	1.65	1.27	0.89	0.65	1.23	0.74	0.76	1.26
3¾×2⅝	3/4	14.30	4.35	5.47	2.15	1.10	0.69	0.59	2.29	1.18	0.81	1.37
	3/8	7.70	2.26	3.16	1.28	1.19	0.75	0.60	1.26	0.66	0.68	1.24
3½×3	13/16	15.7	4.62	4.98	3.33	1.04	0.85	0.62	2.20	1.65	0.98	1.23
	3/4	14.7	4.31	4.70	3.15	1.04	0.85	0.62	2.05	1.54	0.96	1.21
	11/16	13.6	4.00	4.41	2.96	1.05	0.86	0.62	1.91	1.44	0.94	1.19
	5/8	12.5	3.67	4.11	2.76	1.06	0.87	0.62	1.76	1.33	0.92	1.17

PROPERTIES OF STANDARD AND SPECIAL ANGLES.

ANGLES WITH UNEQUAL LEGS.

Continued.



			I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.
Size, inches.	Thickness of metal, inches.	Weight per foot, pounds.	Area of section, sq. ins.	Moments of inertia. I.		Radii of gyration. R.			Section modulus. r.		Perpendicular distances from centre of gravity to back of flanges.	
				Axis AB.	Axis CD.	Axis AB.	Axis CD.	Axis EF.	Axis AB.	Axis CD.	To back of longer flange, l.	To back of shorter flange, d.
3½×3	9/16	11.4	3.34	3.79	2.55	1.07	0.87	0.62	1.61	1.21	0.90	1.15
	1/2	10.2	3.00	3.45	2.33	1.07	0.88	0.62	1.45	1.10	0.88	1.13
	7/16	9.1	2.65	3.10	2.09	1.08	0.89	0.62	1.29	0.98	0.85	1.10
	3/8	7.8	2.30	2.72	1.85	1.09	0.90	0.62	1.13	0.85	0.83	1.08
	5/16	6.6	1.93	2.33	1.58	1.10	0.90	0.63	0.96	0.72	0.81	1.06
3½×2½	11/16	12.4	3.65	4.13	1.72	1.06	0.67	0.53	1.85	0.99	0.77	1.27
	5/8	11.4	3.36	3.85	1.61	1.07	0.69	0.53	1.71	0.92	0.75	1.25
	9/16	10.4	3.06	3.55	1.49	1.08	0.70	0.53	1.56	0.84	0.73	1.23
	1/2	9.4	2.75	3.24	1.36	1.09	0.70	0.53	1.41	0.76	0.70	1.20
	7/16	8.3	2.43	2.91	1.23	1.09	0.71	0.54	1.26	0.68	0.68	1.18
	3/8	7.2	2.11	2.56	1.09	1.10	0.72	0.54	1.09	0.59	0.66	1.16
	5/16	6.1	1.78	2.19	0.94	1.11	0.73	0.54	0.93	0.50	0.64	1.14
	1/4	4.9	1.44	1.80	0.78	1.12	0.74	0.54	0.75	0.41	0.61	1.11
3⅝×2⅛	3/8	6.6	1.99	2.55	0.65	1.13	0.57	0.48	1.09	0.41	0.53	1.28
3½×2	1/4	4.5	1.31	1.66	0.41	1.13	0.56	0.48	0.72	0.21	0.46	1.21

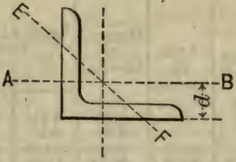
		Weight.	Area.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.
3¼×2	9/16	9.0	2.64	2.64	0.75	1.00	0.53	0.44	1.30	0.53	0.59	1.21
	1/2	8.1	2.38	2.42	0.69	1.01	0.54	0.44	1.17	0.48	0.57	1.19
	3/8	6.2	1.83	1.92	0.55	1.02	0.55	0.44	0.91	0.37	0.52	1.15
	1/4	4.3	1.25	1.36	0.40	1.04	0.57	0.45	0.63	0.26	0.48	1.09
3 ×2½	9/16	9.5	2.78	2.28	1.42	0.91	0.72	0.52	1.15	0.82	0.77	1.02
	1/2	8.5	2.50	2.08	1.30	0.91	0.72	0.52	1.04	0.74	0.75	1.00
	7/16	7.6	2.22	1.88	1.18	0.92	0.73	0.52	0.93	0.66	0.73	0.98
	3/8	6.6	1.92	1.66	1.04	0.93	0.74	0.52	0.81	0.58	0.71	0.96
	5/16	5.5	1.62	1.42	0.90	0.94	0.74	0.53	0.69	0.49	0.68	0.93
	1/4	4.5	1.31	1.17	0.74	0.95	0.75	0.53	0.56	0.40	0.66	0.91
3 ×2	1/2	7.7	2.25	1.92	0.67	0.92	0.55	0.43	1.00	0.47	0.58	1.08
	7/16	6.8	2.00	1.73	0.61	0.93	0.55	0.43	0.89	0.42	0.56	1.06
	3/8	5.9	1.73	1.53	0.54	0.94	0.56	0.43	0.78	0.37	0.54	1.04
	5/16	5.0	1.47	1.32	0.47	0.95	0.57	0.43	0.66	0.32	0.52	1.02
	1/4	4.0	1.19	1.09	0.39	0.95	0.57	0.43	0.54	0.25	0.49	0.99
2½×2	1/2	6.8	2.00	1.14	0.64	0.75	0.56	0.42	0.70	0.46	0.63	0.88
	7/16	6.1	1.78	1.03	0.58	0.76	0.57	0.42	0.62	0.41	0.60	0.85
	3/8	5.3	1.55	0.91	0.51	0.77	0.58	0.42	0.55	0.36	0.58	0.83
	5/16	4.5	1.31	0.79	0.45	0.78	0.58	0.42	0.47	0.31	0.56	0.81
	1/4	3.7	1.06	0.65	0.37	0.78	0.59	0.42	0.38	0.25	0.54	0.79
	3/16	2.8	0.81	0.51	0.29	0.79	0.60	0.43	0.29	0.20	0.51	0.76
2½×1¾	1/4	3.4	1.00	0.62	0.25	0.79	0.50	0.38	0.37	0.20	0.45	0.83
	3/16	2.6	0.76	0.49	0.20	0.80	0.51	0.38	0.29	0.15	0.43	0.81
2½×1½	3/8	4.6	1.36	0.82	0.22	0.78	0.40	0.32	0.52	0.20	0.42	0.92
	3/16	2.4	0.71	0.46	0.13	0.80	0.42	0.33	0.28	0.11	0.35	0.85
2½×1¼	3/8	4.3	1.27	0.77	0.13	0.78	0.32	0.26	0.50	0.14	0.35	0.97
	3/16	2.2	0.67	0.43	0.07	0.80	0.33	0.27	0.27	0.08	0.28	0.91
2¼×1½	1/2	5.5	1.63	0 82	0.26	0.71	0.40	0.39	0.59	0.26	0.48	0.86
	3/8	4.3	1.27	0.61	0.21	0.69	0.41	0.39	0.42	0.20	0.44	0.81
	1/4	3.0	0.88	0.44	0.16	0.71	0.42	0.40	0.30	0.14	0.39	0.77
	3/16	2.3	0.67	0.34	0.12	0.72	0.43	0.40	0.23	0.11	0.37	0.75

ANGLES WITH UNEQUAL LEGS (*Continued*).

			I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.
Size, inches.	Thickness of metal, inches.	Weight per foot, pounds.	Area of section, sq. ins.	Moments of inertia. *I.*		Radii of gyration. *r.*			Section modulus. *R.*		Perpendicular distances from centre of gravity to back of flanges.	
				Axis AB.	Axis CD.	Axis AB.	Axis CD.	Axis EF.	Axis AB.	Axis CD.	To back of longer flange, *l*.	To back of shorter flange, *d*.
2 × 1¾	5/16	3.64	1.07	0.40	0.28	0.62	0.51	0.36	0.30	0.23	0.52	0.65
	3/16	2.28	0.67	0.26	0.19	0.61	0.53	0.36	0.19	0.15	0.47	0.60
2 × 1½	5/16	3.39	1.00	0.38	0.18	0.62	0.42	0.32	0.29	0.17	0.44	0.69
	3/16	2.11	0.62	0.25	0.12	0.63	0.44	0.32	0.18	0.11	0.39	0.64
2 × 1⅜	1/4	2.66	0.78	0.37	0.12	0.63	0.39	0.30	0.23	0.12	0.37	0.69
	3/16	2.03	0.60	0.24	0.09	0.63	0.40	0.31	0.18	0.09	0.35	0.66
2 3/16 × 1 7/16	3/8	3.90	1.19	0.50	0.17	0.65	0.38	0.32	0.36	0.17	0.42	0.80
2 × 1¼	3/16	1.90	0.57	0.23	0.07	0.64	0.35	0.30	0.18	0.07	0.31	0.69
1⅜ × 1⅛	5/16	2.45	0.72	0.13	0.08	0.43	0.33	0.25	0.15	0.10	0.38	0.51
	1/8	1.02	0.30	0.06	0.03	0.43	0.33	0.24	0.06	0.04	0.29	0.42
1⅜ × 1	1/4	1.80	0.53	0.09	0.04	0.41	0.27	0.22	0.09	0.05	0.29	0.48
	1/8	1.00	0.28	0.05	0.02	0.44	0.29	0.22	0.06	0.03	0.26	0.44
1⅜ × ⅞	1/8	0.85	0.25	0.052	0.016	0.46	0.25	0.21	0.057	0.024	0.22	0.47
1 × ⅝	1/8	0.63	0.19	0.019	0.006	0.32	0.17	0.15	0.028	0.012	0.13	0.32

PROPERTIES OF STANDARD AND SPECIAL ANGLES.

ANGLES WITH EQUAL LEGS.



Size, in inches.	Thickness of metal.	W'ght per ft.	I. Area in sq. ins.	II. Moment of inertia. Axis AB.	IV. Radii of gyration. Axis AB.	VI. Radii of gyration. Axis EF.	VII. Section modulus, *R*. Axis AB.	IX. Distance of centre of gravity from back of flange, *d*.
	1 1/8	56.9	16.73	97.97	2.42	1.55	17.53	2.41
	1 1/16	54.0	15.87	93.53	2.43	1.56	16.67	2.39
	1	51.0	15.00	88.98	2.44	1.56	15.80	2.37
	15/16	48.0	14.12	84.33	2.44	1.56	14.91	2.34
	7/8	45.0	13.23	79.58	2.45	1.57	14.01	2.32
8 ×8	13/16	42.0	12.34	74.71	2.46	1.57	13.11	2.30
	3/4	38.9	11.44	69.74	2.47	1.57	12.18	2.28
	11/16	35.8	10.53	64.64	2.48	1.58	11.25	2.25
	5/8	32.7	9.61	59.42	2.49	1.58	10.30	2.23
	9/16	29.5	8.68	54.09	2.50	1.58	9.34	2.21
	1/2	26.4	7.75	48.63	2.50	1.58	8.37	2.19
	1	37.4	11.00	35.46	1.80	1.16	8.57	1.86
	15/16	35.3	10.37	33.72	1.80	1.16	8.11	1.84
	7/8	33.1	9.74	31.92	1.81	1.17	7.64	1.82
	13/16	30.9	9.09	30.06	1.82	1.17	7.15	1.80
	3/4	28.7	8.44	28.15	1.83	1.17	6.66	1.78
6 ×6	11/16	26.5	7.78	26.19	1.83	1.17	6.17	1.75
	5/8	24.2	7.11	24.16	1.84	1.18	5.66	1.73
	9/16	21.9	6.43	22.07	1.85	1.18	5.14	1.71
	1/2	19.6	5.75	19.91	1.86	1.18	4.61	1.68
	7/16	17.2	5.06	17.68	1.87	1.19	4.07	1.66
	3/8	14.8	4.36	15.39	1.88	1.19	3.53	1.64
	1	30.6	9.00	19.64	1.48	0.96	5.80	1.61
	15/16	28.9	8.50	18.71	1.48	0.96	5.49	1.59
	7/8	27.2	7.99	17.75	1.49	0.96	5.17	1.57
	13/16	25.4	7.46	16.77	1.50	0.97	4.85	1.55
	3/4	23.6	6.94	15.74	1.51	0.97	4.53	1.52
5 ×5	11/16	21.8	6.42	14.68	1.51	0.97	4.20	1.50
	5/8	20.0	5.86	13.58	1.52	0.97	3.86	1.48
	9/16	18.1	5.31	12.44	1.53	0.98	3.51	1.46
	1/2	16.2	4.75	11.25	1.54	0.98	3.15	1.43
	7/16	14.3	4.18	10.02	1.55	0.98	2.79	1.41
	3/8	12.3	3.61	8.74	1 56	0.99	2.42	1.39

PROPERTIES OF STANDARD AND SPECIAL ANGLES.

ANGLES WITH EQUAL LEGS (*continued*).

			I.	II.	IV.	VI.	VII.	IX.
Size, in inches.	Thickness of metal.	W'ght per ft.	Area in sq. ins.	Moment of inertia. Axis AB.	Radii of gyration. Axis AB.	Radii of gyration. Axis EF.	Section modulus, *R*. Axis AB.	Distance of center of gravity from back of flange. *d*.
4½×4½	5/8	17.8	5.23	9.71	1.36	0.87	3.09	1.35
	9/16	16.1	4.75	8.91	1.37	0.88	2.81	1.33
	1/2	14.5	4.25	8.07	1.38	0.88	2.53	1.31
	7/16	12.7	3.75	7.20	1.39	0.88	2.24	1.29
	3/8	11.0	3.23	6.30	1.40	0.89	1.95	1026
	5/16	9.2	2.71	5.36	1.40	0.89	1.64	1.24
4 ×4	13/16	19.9	5.84	8.14	1.18	0.77	3.01	1.29
	3/4	18.5	5.44	7.67	1.19	0.77	2.81	1.27
	11/16	17.1	5.03	7.17	1.19	0.77	2.61	1.25
	5/8	15.7	4.61	6.66	1.20	0.77	2.40	1.23
	9/16	14.3	4.18	6.12	1.21	0.78	2.19	1.21
	1/2	12.8	3.75	5.56	1.22	0.78	1.97	1.18
	7/16	11.3	3.31	4.97	1.23	0.78	1.75	1.16
	3/8	9.8	2.86	4.36	1.23	0.79	1.52	1.14
	5/16	8.2	2.40	3.71	1.24	0.79	1.29	1.12
3½×3½	13/16	17.1	5.03	5.25	1.02	0.67	2.25	1.17
	3/4	16.0	4.69	4.96	1.03	0.67	2.11	1.15
	11/16	14.8	4.34	4 65	1.04	0.67	1.96	1.12
	5/8	13.6	3.98	4.33	1.04	0.67	1.81	1.10
	9/16	12.3	3.62	3.99	1.05	0.68	1.65	1.08
	1/2	11.1	3.25	3 64	1.06	0.68	1.49	1.06
	7/16	9.8	2.87	3.26	1.07	0.68	1.32	1.04
	3/8	8.5	2.48	2.87	1.07	0.69	1.15	1.01
	5/16	7.1	2.09	2.45	1.08	0.69	0.98	0.99
3¼×3¼	3/4	14.7	4.32	2.96	0.79	0.53	1.36	1.08
	3/8	7.8	2.29	2.27	0.99	0.66	0.99	0.95
3 ×3	5/8	11.4	3.36	2.62	0.88	0.57	1.30	0.98
	9/16	10.4	3.06	2.43	0.89	0.58	1.19	0.95
	1/2	9.4	2.75	2.22	0.90	0.58	1.07	0.93
	7/16	8.3	2.43	1.99	0.91	0.58	0.95	0.91
	3/8	7.2	2.11	1.76	0.91	0.58	0.83	0.89
	5/16	6.1	1.78	1.51	0.92	0.59	0.71	0.87
	1/4	4.9	1.44	1.24	0.93	0.59	0.58	0.84
2¾×2¾	1/2	8.5	2.50	1.67	0.82	0.52	0.89	0.87
	7/16	7.6	2.22	1.51	0.82	0.53	0.79	0.85
	3/8	6.6	1.92	1.33	0.83	0.53	0.69	0.82
	5/16	5.5	1.62	1.15	0.84	0.54	0.59	0.80
	1/4	4.5	1.31	0.93	0.85	0.55	0.48	0.78
2½×2½	1/2	7.7	2.25	1.23	0.74	0.47	0.73	0.81
	7/16	6.8	2.00	1.11	0.74	0.48	0.65	0.78
	3/8	5.9	1.73	0.98	0.75	0.48	0.57	0.76
	5/16	5.0	1.47	0.85	0.76	0.49	0.48	0.74
	1/4	4.1	1.19	0.70	0.77	0.49	0.40	0.72
	3/16	3.1	0.90	0.55	0.78	0.49	0.30	0.69

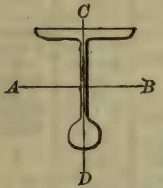
PROPERTIES OF STANDARD AND SPECIAL ANGLES.

ANGLES WITH EQUAL LEGS (*continued*).

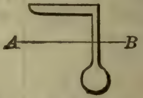
			I.	II.	IV.	VI.	VII.	IX.
Size, in inches.	Thickness of metal.	W'ght per ft.	Area in sq. ins.	Moment of inertia. Axis AB.	Radii of gyration. Axis AB.	Radii of gyration. Axis EF.	Section modulus, *R*. Axis AB.	Distance of centre of gravity from back of flange, *d*.
2¼×2¼	1/2	6.8	2.00	0.87	0.66	0.43	0.58	0.74
	7/16	6.1	1.78	0.79	0.67	0.43	0.52	0.72
	3/8	5.3	1.55	0.70	0.67	0.43	0.45	0.70
	5/16	4.5	1.31	0.61	0.68	0.44	0.39	0.68
	1/4	3.7	1.06	0.51	0.69	0.44	0.32	0.66
	3/16	2.8	0.81	0.39	0.70	0.44	0.24	0.63
2 ×2	7/16	5.3	1.56	0.54	0.59	0.39	0.40	0.66
	3/8	4.7	1.36	0.48	0.59	0.39	0.35	0.64
	5/16	4.0	1.15	0.42	0.60	0.39	0.30	0.61
	1/4	3.2	0.94	0.35	0.61	0.39	0.25	0.59
	3/16	2.5	0.72	0.28	0.62	0.40	0.19	0.57
1¾×1¾	7/16	4.6	1.30	0.35	0.51	0.33	0.30	0.59
	3/8	4.0	1.17	0.31	0.51	0.34	0.26	0.57
	5/16	3.4	1.00	0.27	0.52	0.34	0.23	0.55
	1/4	2.8	0.81	0.23	0.53	0.34	0.19	0.53
	3/16	2.1	0.62	0.18	0.54	0.35	0.14	0.51
1½×1½	3/8	3.4	0.99	0.19	0.44	0.29	0.19	0.51
	5/16	2.9	0.84	0.16	0.44	0.29	0.162	0.49
	1/4	2.4	0.69	0.14	0.45	0.29	0.134	0.47
	3/16	1.8	0.53	0.11	0.46	0.29	0.104	0.44
	1/8	1.2	0.36	0.08	0.46	0.30	0.070	0.42
1¼×1¼	5/16	2.4	0.69	0.09	0.36	0.23	0.109	0.42
	1/4	1.9	0.56	0.077	0.37	0.24	0.091	0.40
	3/16	1.5	0.43	0.061	0.38	0.24	0.071	0.38
	1/8	1.0	0.30	0.044	0.38	0.25	0.049	0.35
1 ×1	1/4	1.5	0.44	0.037	0.29	0.19	0.056	0.34
	3/16	1.2	0.34	0.030	0.30	0.19	0.044	0.32
	1/8	0.8	0.24	0.022	0.31	0.20	0.031	0.30
⅞×⅞	3/16	1.0	0.29	0.019	0.26	0.18	0.033	0.29
	1/8	0.7	0.21	0.014	0.26	0.19	0.023	0.26
¾×¾	3/16	0.8	0.25	0.012	0.22	0.16	0.024	0.26
	1/8	0.6	0.17	0.009	0.23	0.17	0.017	0.23

PROPERTIES OF CARNEGIE DECK-BEAMS AND BULB ANGLES.

DECK-BEAMS.—STEEL.



Deck-beam.



Bulb angle.

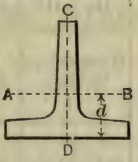
Depth of beam.	Weight per foot, lbs.	Thickness of web, in.	Width of flange, ins.	I. Area of section, sq. ins.	II. Moments of inertia, I. Axis AB.	III. Moments of inertia, I. Axis CD.	IV. Radii of gyration, r. Axis AB.	V. Radii of gyration, r. Axis CD.	VII. Section modulus, neutral axis, AB. R.
11.5″	37.00	.55	5.30	10.9	194.7	6.60	4.23	0.78	30.6
11.5″	32.20	.42	5.17	9.5	178.7	6.06	4.34	0.80	27.6
10″	35.70	.63	5.50	10.5	139.9	7.41	3.64	0.84	25.7
10″	27.23	.38	5.25	8.0	118.4	6.12	3.83	0.87	21.2
9″	30.00	.57	5.07	8.8	93.2	5.18	3.25	0 75	19.6
9″	26.00	.44	4.94	7.6	85.2	4.61	3.35	0.76	17.7
8″	24.48	.47	5.16	7.2	62.8	4.45	2.97	0.79	14.1
8″	20.15	.31	5.00	5.9	55.6	3.90	3.08	0.82	12.2
7″	23.46	.54	5.10	6.9	45.5	4.30	2.57	0.79	11.7
7″	18.11	.31	4.87	5.3	38.8	3.55	2.70	0.82	9.7
6″	17.16	.43	4.53	5.0	24.4	2.66	2.20	0.73	7.2
6″	14.10	.28	4.38	4.1	21.6	2.22	2.28	0.72	6.1

BULB ANGLES.—STEEL.

10″	32.00	.63	3.5	9.41	116.0	.....	3.51	.....	21.6
10″	26.50	.48	3.5	7.80	104.2	.....	3.66	.....	19.9
9″	21.80	.44	3.5	6.41	69.3	.....	3.33	.....	14.5
8″	19.23	.41	3.5	5.66	48.8	.....	2.95	.....	11.7
7″	18.25	.44	3.0	5.37	34.9	.....	2.56	.....	9.6
7″	16.00	.34	3.0	4.71	32.2	.....	6.61	.....	8.7
6″	17.20	.50	3.0	5.06	23.9	.....	2.16	.....	7.6
6″	13.75	.38	3.0	4.04	20.1	.....	2.21	.....	6.6
6″	12.30	.31	3.0	3.62	18.6	.....	2.28	.....	5.7
5″	10.00	.31	2.5	2.94	10.2	.....	1.86	.....	4.1

PROPERTIES OF CARNEGIE T SHAPES.—STEEL.

Thickness varies slightly, that given being the minimum.

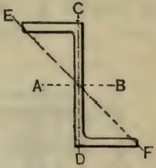


Size: Flange by stem, ins.	Thickness of flange, in.	Weight per foot, lbs.	I. Area, sq. ins.	II. Moments of inertia, *I*. Axis AB.	III. Moments of inertia, *I*. Axis CD.	IV. Radii of gyration, *r*. Axis AB.	V. Radii of gyration, *r*. Axis CD.	VII. Section modulus, *R*. Axis AB.	VIII. Section modulus, *R*. Axis CD.	IX. Base to neutral axis, *d*.
5 × 3	1/2	13.6	3.99	2.6	5.6	0.82	1.19	1.18	2.22	0.75
5 × 2½	3/8	11.0	3.24	1.6	4.3	0.71	1.16	0.86	1.70	0.65
4½ × 3½	7/16	15.8	4.65	5.1	3.7	1.04	0.90	2.13	1.65	1.11
4½ × 3	5/16	18.5	2.55	1.8	2.6	0.87	1.03	0.81	1.16	0.73
4½ × 3	3/8	10.0	3.00	2.1	3.1	0.86	1.04	0.94	1.38	0.75
4½ × 2½	5/16	8.0	2.40	1.1	2.6	0.69	1.07	0.56	1.16	0.58
4½ × 2½	3/8	9.3	2.79	1.2	3.1	0.68	1.08	0.65	1.38	0.60
4 × 5	1/2	15.6	4.56	10.7	2.8	1.54	0.79	3.10	1.41	1.56
4 × 5	3/8	12.0	3.54	8.5	2.1	1.56	0.78	2.43	1.06	1.51
4 × 4½	1/2	14.6	4.29	8.0	2.8	1.37	0.81	2.55	1.41	1.37
4 × 4½	3/8	11.4	3.36	6.3	2.1	1.38	0.80	1.98	1.06	1.31
4 × 4	1/2	13.7	4.02	5.7	2.8	1.20	0.84	2.02	1.40	1.18
4 × 4	3/8	10.9	3.21	4.7	2.2	1.23	0.84	1.64	1.09	1.15
4 × 3	3/8	9.3	2.73	2.0	2.1	0.86	0.88	0.88	1.05	0.78
4 × 2½	3/8	8.6	2.52	1.2	2.1	0.69	0.92	0.62	1.05	0.63
4 × 2½	5/16	7.3	2.16	1.0	1.8	0.70	0.91	0.55	0.88	0.60
4 × 2	3/8	7.9	2.31	0.60	2.1	0.52	0.96	0.40	1.05	0.48
4 × 2	5/16	6.6	1.95	0.54	1.8	0.51	0.95	0.34	0.88	0.51
3½ × 4	1/2	12.8	3.75	5.5	1.89	1.21	0.72	1.98	1.08	1.25
3½ × 4	3/8	9.9	2.91	4.3	1.42	1.22	0.70	1.55	0.81	1.19
3½ × 3½	1/2	11.7	3.45	3.7	1.89	1.04	0.74	1.52	1.08	1.06
3½ × 3½	3/8	9.2	2.70	3.0	1.42	1.05	0.73	1.19	0.81	1.01
3½ × 3	1/2	10.9	3.21	2.4	1.88	0.87	0.77	1.13	1.08	0.88
3½ × 3	3/8	8.5	2.49	1.9	1.41	0.88	0.75	0.88	0.81	0.83
3½ × 3	5/16	7.8	2.28	1.6	1.18	0.89	0.76	0.72	0.68	0.78
3 × 4	1/2	11.8	3.48	5.2	1.21	1.23	0.59	1.94	0.81	1.32
3 × 4	7/16	10.6	3.12	4.8	1.09	1.25	0.60	1.78	0.72	1.32
3 × 4	3/8	9.3	2.73	4.3	0.93	1.26	0.59	1.57	0.62	1.29
3 × 3½	1/2	10.9	3.21	3.5	1.20	1.06	0.62	1.49	0.80	1.12
3 × 3½	7/16	9.8	2.88	3.3	1.31	1.08	0.68	1.37	0.88	1.11

PROPERTIES OF CARNEGIE T SHAPES.—STEEL (*concluded*).

Size: flange by stem, ins.	Thickness of flange, in.	Weight per foot, lbs.	I.	II.	III.	IV.	V.	VII.	VIII.	IX.
			Area, sq. ins.	Moments of inertia, *I*.		Radii of gyration, *r*.		Section modulus, *R*.		Base to neutral axis, *d*.
				Axis AB.	Axis CD.	Axis AB.	Axis CD.	Axis AB.	Axis CD.	
3 ×3½	3/8	8.5	2.49	2.9	0.93	1.09	0.61	1.21	0.62	1.09
3 ×3	1/2	10.0	2.94	2.3	1.20	0.88	0.64	1.10	0.80	0.93
3 ×3	7/16	9.1	2.67	2.1	1.08	0.90	0.64	1.01	0.72	0.92
3 ×3	3/8	7.8	2.28	1.8	0.90	0.90	0.63	0.86	0.60	0.88
3 ×3	5/16	6.6	1.95	1.6	0.75	0.90	0.62	0.74	0.50	0.86
3 ×2½	3/8	7.2	2.10	1.1	0.89	0.72	0.66	0.60	0.60	0.71
3 ×2½	5/16	6.1	1.80	0.94	0.75	0.73	0.65	0.52	0.50	0.68
2¾×2	5/16	7.4	2.16	1.1	0.62	0.71	0.54	0.75	0.45	0.53
2½×3	3/8	7.2	2.10	1.8	0.54	0.92	0.51	0.87	0.43	0.97
2½×3	5/16	6.1	1.80	1.6	0.44	0.94	0.51	0.76	0.35	0.92
2½×2¾	3/8	6.7	1.98	1.4	0.66	0.84	0.58	0.73	0.53	0.87
2½×2¾	5/16	5.8	1.71	1.2	0.44	0.83	0.51	0.60	0.35	0.83
2½×2½	3/8	6.4	1.89	1.0	0.52	0.74	0.53	0.59	0.42	0.76
2½×2½	5/16	5.5	1.62	0.87	0.44	0.74	0.52	0.50	0.35	0.74
2½×1¼	3/16	2.9	0.84	0.09	0.29	0.31	0.58	0.09	0.23	0.29
2¼×2¼	5/16	4.9	1.44	0.66	0.33	0.68	0.48	0.42	0.30	0.69
2¼×2¼	1/4	4.1	1.20	0.51	0.25	0.67	0.47	0.32	0.22	0.66
2 ×2	5/16	4.3	1.26	0.45	0.23	0.60	0.43	0.33	0.23	0.63
2 ×2	1/4	3.7	1.08	0.36	0.18	0.60	0.42	0.25	0.18	0.59
2 ×1½	1/4	3.1	0.90	0.16	0.18	0.42	0.45	0.15	0.18	0.42
1¾×1¾	1/4	3.1	0.90	0.23	0.12	0.51	0.37	0.19	0.14	0.54
1¾×1¼	3/8	3.6	1.05	0.12	0.19	0.33	0.41	0.15	0.22	0.91
1½×1½	1/4	2.4	0.75	0.15	0.08	0.49	0.34	0.14	0.10	0.42
1½×1½	3/16	1.84	0.54	0.11	0.06	0.45	0.31	0.11	0.07	0.44
1¼×1¼	1/4	2.04	0.60	0.08	0.05	0.36	0.27	0.10	0.07	0.40
1¼×1¼	3/16	1.53	0.45	0.06	0.03	0.37	0.26	0.07	0.05	0.38
1 ×1	3/16	1.23	0.36	0.03	0.02	0.29	0.21	0.05	0.04	0.32
1 ×1	1/8	0.87	0.26	0.02	0.01	0.29	0.21	0.03	0.02	0.29

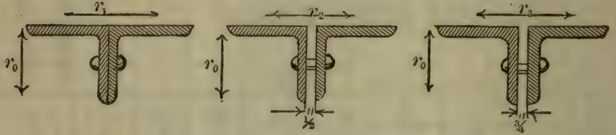
PROPERTIES OF CAMBRIA AND CARNEGIE STEEL Z-BARS.



Depth of web, in.	Width of flange, in.	Thickness of metal, in.	Weight per foot, lbs.	I. Area of section, sq. ins.	II. Moments of inertia. *I.* Axis AB.	III. Moments of inertia. *I.* Axis CD.	IV. Radii of gyration. *r.* Axis AB.	V. Radii of gyration. *r.* Axis CD.	VI. Radii of gyration. *r.* Axis EF.
6	3 1/2	3/8	15.6	4.59	25.32	9.11	2.35	1.41	0.83
6 1/16	3 9/16	7/16	18.3	5.39	29.80	10.95	2.35	1.43	0.84
6 1/8	3 5/8	1/2	21.0	6.19	34.36	12.87	2.36	1.44	0.84
6	3 1/2	9/16	22.7	6.68	34.64	12.59	2.28	1.37	0.81
6 1/16	3 9/16	5/8	25.4	7.46	38.86	14.42	2.28	1.39	0.82
6 1/8	3 5/8	11/16	28.0	8.25	43.18	16.34	2.29	1.41	0.84
6	3 1/2	3/4	29.3	8.63	42.12	15.44	2.21	1.34	0.81
6 1/16	3 9/16	13/16	32.0	9.40	46.13	17.27	2.22	1.36	0.82
6 1/8	3 5/8	7/8	34.6	10.17	50.22	19.18	2.22	1.37	0.83
5	3 1/4	5/16	11.6	3.40	13.36	6.18	1.98	1.35	0.75
5 1/16	3 5/16	3/8	13.9	4.10	16.18	7.65	1.99	1.37	0.76
5 1/8	3 3/8	7/16	16.4	4.81	19.07	9.20	1.99	1.38	0.77
5	3 1/4	1/2	17.8	5.25	19.19	9.05	1.91	1.31	0.74
5 1/16	3 5/16	9/16	20.2	5.94	21.83	10.51	1.91	1.33	0.75
5 1/8	3 3/8	5/8	22.6	6.64	24.53	12.06	1.92	1.35	0.76
5	3 1/4	11/16	23.7	6.96	23.68	11.37	1.84	1.28	0.73
5 1/16	3 5/16	3/4	26.0	7.64	26.16	12.83	1.85	1.30	0.75
5 1/8	3 3/8	13/16	28.3	8.33	28.70	14.36	1.86	1.31	0.76
4	3 1/16	1/4	8.2	2.41	6.28	4.23	1.62	1.33	0.67
4 1/16	3 1/8	5/16	10.3	3.03	7.94	5.46	1.62	1.34	0.68
4 1/8	3 3/16	3/8	12.4	3.66	9.63	6.77	1.62	1.36	0.69
4	3 1/16	7/16	13.8	4.05	9.66	6.73	1.55	1.29	0.66
4 1/16	3 1/8	1/2	15.8	4.66	11.18	7.96	1.55	1.31	0.67
4 1/8	3 3/16	9/16	17.9	5.27	12.74	9.26	1.55	1.33	0.69
4	3 1/16	5/8	18.9	5.55	12.11	8.73	1.48	1.25	0.66
4 1/16	3 1/8	11/16	20.9	6.14	13.52	9.95	1.48	1.27	0.67
4 1/8	3 3/16	3/4	22.9	6.75	14.97	11.24	1.49	1.29	0.69
3	2 11/16	1/4	6.7	1.97	2.87	2.81	1.21	1.19	0.55
3 1/16	2 3/4	5/16	8.4	2.48	3.64	3.64	1.21	1.21	0.56
3	2 11/16	3/8	9.7	2.86	3.85	3.92	1.16	1.17	0.55
3 1/16	2 3/4	7/16	11.4	3.36	4.57	4.75	1.17	1.19	0.56
3	2 11/16	1/2	12.5	3.69	4.59	4.85	1.12	1.15	0.55
3 1/16	2 3/4	9/16	14.2	4.18	5.26	5.70	1.12	1.17	0.56

TABLE A.—RADII OF GYRATION FOR A PAIR OF ANGLES PLACED BACK TO BACK.

ANGLES WITH EQUAL LEGS.



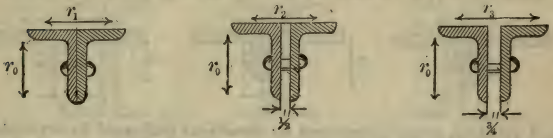
Radii of gyration given correspond to directions indicated by arrow-heads.

Size, in inches.	Weight per foot of single angle, in lbs.	*Area of section, in ins.	Radii of gyration. r_0.	r_1.	r_2.	r_3.
8 ×8 ×½	26.4	15.50	2.50	3.32	3.49	3.58
8 ×8 ×1⅛	56.9	33.46	2.42	3.42	3.60	3.69
6 ×6 × ⅜	14.8	8.72	1.88	2.49	2.67	2.76
6 ×6 × ½	19.6	11.50	1.86	2.52	2.70	2.80
6 ×6 × ¾	28.7	16.88	1.83	2.55	2.73	2.83
6 ×6 ×1	37.4	22.00	1.80	2.59	2.77	2.87
5 ×5 × ⅜	12.3	7.22	1.56	2.09	2.26	2.35
5 ×5 × ½	16.2	9.50	1.54	2.11	2.29	2.38
5 ×5 × ¾	23.6	13.88	1.51	2.15	2.33	2.43
5 ×5 ×1	30.6	18.00	1.48	2.19	2.38	2.48
4 ×4 × 5/16	8.2	4.80	1.24	1.67	1.85	1.94
4 ×4 × ⅜	9.8	5.72	1.23	1.69	1.88	1.97
4 ×4 × ⅝	15.7	9.22	1.20	1.72	1.91	2.00
4× 4 × 13/16	19.9	11.68	1.18	1.75	1.94	2.04
3½×3½× 5/16	7.1	4.18	1.08	1.47	1.65	1.74
3½×3½× ⅜	8.5	4.96	1.07	1.49	1.67	1.77
3½×3½× ⅝	13.6	7.96	1.04	1.52	1.70	1.81
3½×3½× 13/16	17.1	10.06	1.02	1.55	1.74	1.85
3 ×3 × ¼	4.9	2.88	0.93	1.25	1.43	1.53
3 ×3 × ⅜	7.2	4.22	0.91	1.27	1.45	1.56
3 ×3 × ½	9.4	5.50	0.90	1.29	1.48	1.59
3 ×3 × ⅝	11.4	6.72	0.88	1.32	1.51	1.62
2¾×2¾× ¼	4.5	2.62	0.85	1.15	1.34	1.44
2¾×2¾× ½	8.5	5.00	0.82	1.19	1.39	1.49
2½×2½× 3/16	3.1	1.80	0.78	1.04	1.22	1.32
2½×2½× ½	7.7	4.50	0.74	1.10	1.29	1.40
2¼×2¼× 3/16	2.8	1.62	0.70	0.94	1.12	1.23
2¼×2¼× ½	6.8	4.00	0.66	0.99	1.19	1.30
2 ×2 × ¼	3.2	1.88	0.61	0.85	1.03	1.14

* The figures in this column give the area of both angles.

TABLE B.—RADII OF GYRATION FOR A PAIR OF ANGLES PLACED BACK TO BACK.

ANGLES WITH UNEQUAL LEGS—LONG LEG VERTICAL.



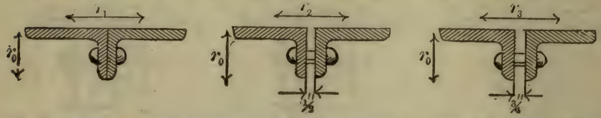
Radii of gyration given correspond to directions indicated by arrow-heads.

Size, in inches.	Weight per foot of single angle, in lbs.	†Area of section, in ins.	Radii of gyration.			
			r_0.	r_1.	r_2.	r_3.
*8 × 6 × ½	23.0	13.52	2.56	2.32	2.49	2.57
*8 × 6 × 1	45.6	26.82	2.53	2.47	2.65	2.74
6 × 4 × ⅜	12.3	7.22	1.93	1.50	1.67	1.76
6 × 4 × 13/16	25.4	14.94	1.87	1.55	1.74	1.84
6 × 3½ × ⅜	11.7	6.84	1.94	1.26	1.43	1.53
6 × 3½ × ½	15.3	9.00	1.92	1.28	1.46	1.56
6 × 3½ × ⅝	18.9	11.10	1.90	1.30	1.49	1.59
6 × 3½ × 13/16	24.0	14.12	1.88	1.33	1.52	1.62
5 × 4 × ⅜	11.0	6.46	1.59	1.58	1.75	1.85
5 × 4 × ¾	21.1	12.38	1.54	1.62	1.81	1.91
5 × 3½ × ⅜	10.4	6.10	1.60	1.33	1.51	1.60
5 × 3½ × ¾	19.8	11.62	1.55	1.39	1.59	1.68
5 × 3 × ⅜	9.8	5.72	1.61	1.10	1.27	1.37
5 × 3 × ½	12.8	7.50	1.59	1.12	1.30	1.39
5 × 3 × ⅝	15.7	9.22	1.57	1.14	1.33	1.42
5 × 3 × ¾	18.5	10.88	1.55	1.17	1.36	1.46
4 × 3½ × ⅜	9.1	5.34	1.25	1.43	1.60	1.70
4 × 3½ × ¾	17.2	10.12	1.20	1.44	1.62	1.72
4 × 3 × ⅜	8.5	4.96	1.26	1.46	1.64	1.74
4 × 3 × ¾	16.0	9.38	1.22	1.48	1.67	1.77
3½ × 2½ × ¼	4.9	2.88	1.12	0.96	1.13	1.23
3½ × 2½ × ⅜	7.2	4.22	1.10	0.98	1.16	1.26
3½ × 2½ × ½	9.4	5.50	1.09	1.00	1.19	1.29
3½ × 2½ × 11/16	12.4	7.30	1.06	1.03	1.23	1.33
3 × 2 × ¼	4.0	2.38	0.95	0.75	0.93	1.03
3 × 2 × ½	7.7	4.50	0.92	0.80	1.00	1.10
2½ × 2 × 5/16	2.8	1.62	0.79	0.79	0.97	1.07
2½ × 2 × ½	6.8	4.00	0.75	0.84	1.04	1.15

* Rolled only by the Pencoyd Iron Co. Works.
† The figures in this column give the area of both angles.

TABLE C.—RADII OF GYRATION FOR A PAIR OF ANGLES PLACED BACK TO BACK.

ANGLES WITH UNEQUAL LEGS.—SHORT LEG VERTICAL.

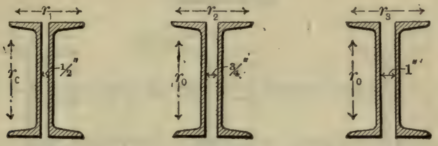


Radii of gyration given correspond to directions indicated by arrow-heads.

Size in inches.	Weight per foot of single angle, in lbs.	* Area of section in inches.	Radii of gyration.			
			r_9.	r_1.	r_2.	r_3.
6 × 4 × 3/8	12.3	7.22	1.17	2.74	2.92	3.01
6 × 4 × 1/2	16.2	9.50	1.15	2.76	2.94	3.04
6 × 4 × 3/4	23.6	13.88	1.12	2.80	2.99	3.09
6 × 4 × 1	30.6	18.00	1.09	2.85	3.04	3.14
6 × 3½ × 3/8	11.7	6.84	0.99	2.81	3.00	3.10
6 × 3½ × 1	28.9	17.00	0.92	2.93	3.13	3.23
5 × 4 × 3/8	11.0	6.46	1.20	2.20	2.38	2.48
5 × 4 × 7/8	24.2	14.22	1.14	2.29	2.48	2.58
5 × 3½ × 5/16	8.7	5.12	1.03	2.26	2.44	2.54
5 × 3½ × 7/8	22.7	13.34	0.96	2.36	2.55	2.65
5 × 3 × 5/16	8.2	4.80	0.85	2.33	2.51	2.61
5 × 3 × 13/16	19.9	11.68	0.80	2.42	2.62	2.72
4 × 3½ × 5/16	7.7	4.50	1.07	1.73	1.91	2.00
4 × 3½ × 1/2	11.9	7.00	1.04	1.76	1.95	2.04
4 × 3½ × 5/8	14.6	8.60	1.03	1.78	1.98	2.07
4 × 3½ × 13/16	18.5	10.86	1.01	1.81	2.01	2.11
4 × 3 × 5/16	7.1	4.18	0.89	1.79	1.97	2.07
4 × 3 × 13/16	17.1	10.06	0.83	1.88	2.08	2.18
3½ × 3 × 5/16	6.6	3.86	0.90	1.52	1.71	1.80
3½ × 3 × 13/16	15.7	9.24	0.85	1.61	1.81	1.91
3½ × 2½ × 1/4	4.9	2.88	0.74	1.58	1.76	1.86
3½ × 2½ × 11/16	12.4	7.30	0.67	1.66	1.86	1.96
3 × 2½ × 1/4	4.5	2.62	0.75	1.31	1.50	1.59
3 × 2½ × 9/16	9.5	5.56	0.72	1.37	1.56	1.66
3 × 2 × 1/4	4.0	2.38	0.57	1.38	1.56	1.66
3 × 2 × 1/2	7.7	4.50	0.55	1.42	1.62	1.73
2½ × 2 × 3/16	2.8	1.62	0.60	1.10	1.28	1.39
2½ × 2 × 1/2	6.8	4.00	0.56	1.16	1.35	1.46

* The figures in this column give the area of both angles.

TABLE D.—RADII OF GYRATION FOR A PAIR OF STANDARD CHANNELS PLACED BACK TO BACK.

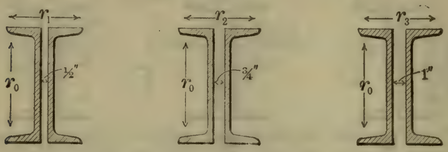


½″ ¾″ 1″

Radii of gyration given correspond to directions indicated by arrow-heads.

Depth in inches.	Thickness of web.	Weight per foot of one channel.	Area of two channels.	Radii of gyration.			
				r_0.	r_1.	r_2.	r_3.
15	0.40	33.00	19.80	5.62	1.38	1.48	1.58
	0.43	35.00	20.58	5.58	1.38	1.47	1.57
	0.52	40.00	23.52	5.43	1.37	1.46	1.56
	0.62	45.00	26.48	5.32	1.37	1.45	1.56
	0.72	50.00	29.42	5.23	1.37	1.46	1.56
	0.82	55.00	32.36	5.16	1.38	1.47	1.58
12	0.28	20.50	12.06	4.61	1.24	1.34	1.44
	0.39	25.00	14.70	4.43	1.21	1.31	1.41
	0.51	30.00	17.64	4.28	1.20	1.30	1.40
	0.64	35.00	20.58	4.17	1.21	1.31	1.41
	1.76	40.00	23.52	.09	1.23	1.32	1.43
10	0.24	15.00	8.92	3.87	1.14	1.24	1.34
	0.38	20.00	11.76	3.66	1.10	1.20	1.31
	0.53	25.00	14.70	3.52	1.10	1.20	1.31
	0.68	30.00	17.64	3.42	1.12	1.22	1.33
	0.82	35.00	20.58	3.35	1.16	1.26	1.37
9	0.23	13.25	7.78	3.49	1.09	1.19	1.29
	0.29	15.00	8.82	3.40	1.07	1.17	1.28
	0.45	20.00	11.76	3.21	1.05	1.15	1.26
	0.62	25.00	14.70	3.10	1.07	1.17	1.28

TABLE D.—RADII OF GYRATION FOR A PAIR OF STANDARD CHANNELS PLACED BACK TO BACK (*continued*).



½″ ¾″ 1″

Radii of gyration given correspond to directions indicated by arrow-heads.

Depth in inches.	Thickness of web.	Weight per foot of one channel.	Area of two channels.	Radii of gyration.			
				r_0.	r_1.	r_2.	r_3.
8	0.22	11.25	6.70	3.11	1.04	1.14	1.25
8	0.31	13.75	8.08	2.98	1.04	1.14	1.25
8	0.40	16.25	9.56	2.89	1.03	1.14	1.24
8	0.49	18.75	11.02	2.82	1.03	1.14	1.24
8	0.58	21.25	12.50	2.77	1.03	1.14	1.24
7	0.21	9.75	5.70	2.72	0.99	1.09	1.20
7	0.32	12.25	7.20	2.59	0.99	1.09	1.20
7	0.42	14.75	8.68	2.50	0.99	1.10	1.21
7	0.53	17.25	10.14	2.44	1.00	1.10	1.21
7	0.63	19.75	11.62	2.39	1.00	1.10	1.22
6	0.20	8.00	4.76	2.34	0.94	1.05	1.15
6	0.32	10.50	6.18	2.21	0.94	1.05	1.16
6	0.44	13.00	7.64	2.13	0.95	1.06	1.16
6	0.56	15.50	9.12	2.07	0.95	1.06	1.17
5	0.19	6.50	3.90	1.95	0.89	1.00	1.10
5	0.33	9.00	5.30	1.83	0.90	1.00	1.11
5	0.48	11.50	6.76	1.75	0.91	1.01	1.12
4	0.18	5.25	3.10	1.56	0.84	0.95	1.06
4	0.25	6.25	3.68	1.51	0.84	0.95	1.06
4	0.32	7.25	4.26	1.46	0.84	0.95	1.06
3	0.17	4.00	2.38	1.17	0.80	0.91	1.02
3	0.26	5.00	2.94	1.12	0.81	0.92	1.03
3	0.36	6.00	3.52	1.08	0.83	0.93	1.05

CHAPTER XI.

RESISTANCE TO TENSION.

PHYSICAL PROPERTIES AND SPECIFICATIONS OF IRON AND STEEL.

STRENGTH OF RODS, ROPES, AND CABLES. PROPORTIONS OF UPSET SCREW ENDS, EYE-BARS, TURNBUCKLES, ETC.

THE resistance which any material offers to being pulled apart is due to the tenacity of its fibres, or the cohesion of the particles of which it is composed.

It is evident that the amount of resistance to tension which any cross-section of a body will exert depends only upon the tenacity of its fibres, or the cohesion of its particles, and upon the number of fibres or particles in the cross-section.

As the number of the fibres, or particles, in the section is proportional to the area, the strength of any piece of material must be as the area of its cross-section; and hence, if we know the tenacity of the material per square inch of cross-section, we can obtain the total strength by multiplying it by the area of the section in inches.

The tenacity of different building materials per square inch has been found by pulling apart a bar of the material of known dimensions, and dividing the breaking force by the area of the cross-section of the bar.

Table I. gives the allowable safe values for the tenacity of building materials, as recommended by the best authorities.

Knowing the tenacity of one square inch of the material all that is necessary to determine the tenacity of a piece of any uniform size is to multiply the area of its cross-section, in square inches, by the number in the table opposite the name of the material; or:

For a rectangular bar,

$$\text{Safe load in lbs.} = \text{breadth} \times \text{depth} \times T. \qquad (1)$$

For a round bar,

$$\text{Safe load in lbs.} = 0.7854 \times \text{diameter squared} \times T. \qquad (2)$$

If the size of the bar is desired we have

$$\text{Area of cross-section} = \frac{\text{load}}{T}. \tag{3}$$

For a round bar,

$$\text{Diameter} = \sqrt{\frac{\text{load}}{.7854 \times T}}. \tag{4}$$

T = values for the material, given in Table I.

TABLE I.

Allowable Safe Tensile Stress per Square Inch for Building Materials.

Material.	Safe Strength in lbs. per sq. in. T
Cement, natural, one week old	60 to 100
Cement, Portland, one week, fair average	350
METALS.	
Cast iron	3,000
Copper, cast	2,000
" forged or rolled	5,500
" wire	9,000
Wrought iron *	10,000 to 14,000
Wrought steel, soft	12,000 to 16,000
" " medium †	12,500 to 18,000
Steel wire	20,000
WOODS. (Factor of safety of five to six.) ‡	
Ash, white	2,000
Ash, brown	1,500
Chestnut	1,500
Hemlock	1,200
Oak, white	2,000
Pine, Georgia yellow	2,000
Pine, Oregon (Douglas Fir)	1,800
Pine, Norway	1,600
Pine, white	1,400
Redwood	800
Spruce	1,600
Whitewood	1,200

* See page 324. † See page 331.

‡ The Building Law for Greater New York fixes the permissible unit stress in yellow pine at 1200 lbs. per sq. inch; in oak at 1000 lbs.; in white pine and spruce at 800 lbs., and in hemlock at 600 lbs. These values are about one-tenth of the ultimate strength.

EXAMPLE I.—The strain in the tie-beam of a truss has been found to be 120,000 lbs. What should be the size of the beam, if made of white pine?

Ans. By formula (3) we have

$$\text{Area of cross-section} = \frac{120,000}{1400} = 85.7 \text{ sq. inches.}$$

If we make the depth of the beam 12 inches then the thickness must be $\frac{85.7}{12}$, or 7.2 inches. As the beam is horizontal its own weight would produce an additional strain in the fibres, for which some allowance must be made.

Allowance must also be made for any cutting of the beam or for holes for truss rods. If there is a two-inch hole through the beam, we should use a 10×12 inch beam, which will allow for the hole, and for the weight of the beam.

For the calculation of tie-beams subject to a transverse load see Chapter XV.

EXAMPLE II.—What size angle bar should be used to resist a tensile stress of 60,000 lbs., the material being medium steel?

Ans. Sectional area $= \frac{60,000}{15,000} = 4$ sq. in.

From the tables giving the properties of angles, Chapter X., we find that a $4 \times 4 \times \frac{5}{8}$ angle has an area of 4.61 square inches. This would be reduced by one $\frac{7}{8}$-inch hole, for a $\frac{3}{4}$-inch rivet, which gives the net area $4.61 - \frac{7}{8} \times \frac{5}{8} = 4.06$ square inches, or just above that required.

[The tensile strength of the sizes of angles most commonly used in trusses is given in Table X. For reduction in net area caused by rivet-holes see Table XI., also Table I., Chapter XX.]

Wrought Iron.

Wrought iron is no longer used for the manufacture of structural shapes, such as angles, channels, beams, etc., except in case of special orders, its use in structural work being practically limited to rods, bars, and bolts. Nearly all bolts are made of wrought iron, and truss rods are generally furnished in wrought iron unless steel is specified. Flat tie-bars are made both of iron and steel. The cost of bars and rods is about the same in wrought iron or mild steel, but wrought iron is easier to work than steel.

Tensile Strength and Quality.—The best American rolled iron has a breaking tensile strength of from fifty thousand to sixty thousand pounds per square inch for specimens not exceeding one square inch in section. Ordinary bar iron should not break under a less strain than fifty thousand pounds per square inch, and should not take a *set* under a stress less than twenty-five thousand pounds per square inch. A bar one inch square and one foot long should stretch fifteen per cent. of its length before breaking, and should be capable of being bent, cold, 90° over the edge of an anvil without sign of fracture, and should show a fibrous texture when broken.

Iron that will not meet these requirements is not suitable for structures; but nothing is gained by specifying more severe tests, because, in bars of the sizes and shapes usually required for such work, nothing more can be attained with certainty, and conscientious makers will be unwilling to agree to furnish that which it is not practicable to produce.

The *working strength* of wrought-iron ties in trusses is generally taken at ten thousand pounds per square inch. In places where the load is perfectly steady and constant twelve thousand pounds may be used.

The *extension of iron*, for all practical purposes, is as follows:

Wrought iron, $\frac{1}{10000}$ of its length per ton per square inch.

Cast iron, $\frac{1}{5000}$ of its length per ton per square inch.

Appearance of the Fractured Surface of Wrought Iron.

At one time it was thought that a fibrous fracture was a sign of good tough wrought iron, and that a crystalline fracture showed that the iron was bad, hard, and brittle. Mr. Kirkaldy's experiments, however, show conclusively that, whenever wrought iron breaks suddenly, it invariably presents a crystalline appearance; and, when it breaks gradually, it invariably presents a fibrous appearance. From the same experiments it was also shown that the appearance of the fractured surface of wrought iron is, to a certain extent, an indication of its quality, provided it is known how the stress was applied which produced the fracture.

Small, uniform crystals, of a uniform size and color, or fine, close, silky fibres, indicate a good iron.

Coarse crystals, blotches of color caused by impurities, loose and open fibres, are signs of bad iron; and flaws in the fractured

surface indicate that the piling and welding processes have been imperfectly carried out.

Kirkaldy's Conclusions.*

Mr. David Kirkaldy of England, who made some of the most valuable experiments on record on the strength of wrought iron, came to some conclusions, many of which differed from what had previously been supposed to be true.

The following are of special importance to the student of building construction, and should be carefully studied:

"The breaking-strain does *not* indicate the quality, as hitherto assumed.

"A high breaking-strain may be due to the iron being of superior quality, density, fine, and moderately soft, or simply to its being very hard and unyielding.

"A *low* breaking-strain may be due to looseness and coarseness in the texture; or to extreme softness, although very close and fine in quality.

"The contraction of area at fracture, previously overlooked, forms an essential element in estimating the quality of specimens.

"The respective merits of various specimens can be correctly ascertained by comparing the breaking-strain *jointly* with the contraction of area.

"Inferior qualities show a much greater variation in the breaking-strain than superior.

"Greater differences exist between small and large bars in coarse than in fine varieties.

"The prevailing opinion of a rough bar being stronger than a turned one is erroneous.

"Rolled bars are slightly hardened by being forged down.

"The breaking-strain and contraction of area of iron plates are greater in the direction in which they are rolled than in a transverse direction.

"Iron is less liable to snap the more it is worked and rolled.

"The ratio of ultimate elongation may be greater in short than in long bars, in some descriptions of iron; whilst in others the ratio is not affected by difference in the length.

"Iron, like steel, is softened, and the breaking-strain reduced, by being heated, and allowed to cool slowly.

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* Kirkaldy's Experiments on Wrought Iron and Steel.

"A great variation exists in the strength of iron bars which have been cut and welded. Whilst some bear almost as much as the uncut bar, the strength of others is reduced fully a third.

"The welding of steel bars, owing to their being so easily burned by slightly overheating, is a difficult and uncertain operation.

"Iron is injured by being brought to a white or welding heat, if not at the same time hammered or rolled.

"The breaking strain is considerably less when the strain is applied suddenly instead of gradually, though some have imagined that the reverse is the case.

"The specific gravity is found to generally indicate pretty correctly the quality of specimens.

"The density of iron is *decreased* by the process of wire-drawing and by the similar process of cold rolling,* instead of *increased*, as previously imagined.

"The density of iron is decreased by being drawn out under a tensile strain, instead of increased, as believed by some.

"It must be abundantly evident, from the facts that have been produced, that the *breaking-strain*, when taken alone, gives a false impression of, instead of indicating, the real quality of iron, as the experiments which have been instituted reveal the somewhat *startling* fact, that frequently the inferior kinds of iron actually yield a higher result than the superior. The *reason* of this difference was shown to be due to the fact that, whilst the one quality retained its original area only very slightly decreased by the strain, the *other* was reduced to less than one-half. Now, surely this variation, hitherto unaccountably *completely overlooked*, is of importance as indicating the relative hardness or softness of the material, and thus, it is submitted, forms an essential element in considering the *safe load* that can be practically applied in various structures. *It must be borne in mind* that, although the softness of the material has the effect of lessening the amount of the breaking-strain, it has the very opposite effect as regards the *working-strain*. This holds good for two reasons: first, the softer the iron, the less liable it is to snap; and, second, fine or soft

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* The conclusion of Mr. Kirkaldy in respect to cold rolling is undoubtedly true when the rolling amounts to wire-drawing; but, when the compression of the surface by rolling diminishes the sectional area in greater proportion than it extends the bar, the result, according to the experience of the Pittsburgh manufacturers, is a slight increase in the density of the iron.

iron, being more uniform in quality, can be more *depended upon in practice.* Hence the load which this description of iron can suspend with safety may approach much more nearly the limit of its breaking-strain than can be attempted with the harder or coarser sorts, where a greater margin must necessarily be left.

"As a necessary corollary to what we have just endeavored to establish, the writer now submits, in addition, that the *working-strain* should be in proportion to the breaking-strain per square inch of *fractured area,* and not to the breaking-strain per square inch of *original area,* as heretofore. Some kinds of iron experimented on by the writer will sustain with safety more than double the load that others can sustain, especially in circumstances where the load is unsteady, and the structure exposed to concussions, as in a ship or railway bridge."

Cast Iron.

Cast iron has only about one-third the tensile strength of wrought iron; and as it is liable to air-holes, internal strains from unequal contraction in cooling, and other concealed defects, reducing its effective area for tension, it should never be used where it is subjected to any great tensile stress.

STANDARD SPECIFICATIONS FOR STRUCTURAL CAST IRON.—Except where chilled iron is specified, all castings shall be tough gray iron, free from injurious cold-shuts or blow-holes, true to pattern, and of a workmanlike finish. Sample pieces one inch square, cast from the same heat of metal in sand moulds, shall be capable of sustaining on a clear span of 4 feet 8 inches a central load of 500 pounds when tested in the rough bar.

Structural Steel.*

The strength of structural steel depends largely on the amount of the constituent elements that are associated with the iron, and each of which affect more or less the hardness and strength of the metal.

The principal of these are carbon, manganese, silicon, phosphorus, and sulphur, the first-named being purposely retained as useful or necessary, the others being rejected, as far as practi-

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* Mr. James Christie in "Steel in Construction," published by the A. & P. Roberts Company, proprietors of the Pencoyd Iron Works.

cable, as objectionable when in excess of certain minute proportions.

The grade and character of steel is usually known by the percentage of contained carbon. Steel used in structures usually varies in tensile strength from 55,000 to 70,000 lbs. per square inch of section, or from .10 to .25 per cent. of carbon.

Table II. exhibits the physical characteristics of open-hearth basic steel of the various grades, the results derived from an extensive series of tests indicating the tendency of a total average of the composition hereafter described to approximate to the figures given in table.

The predominant elements other than carbon averaged throughout the series as follows: manganese, .54; phosphorus, .05; sulphur, .05 per cent. Any increase of these elements is attended with an increase of tensile strength and reduced ductility, and *vice versa.* The tensile strength of the steel is also affected to some extent by the temperature at which it is finished, and the rate of cooling, these influences being more apparent in the grades containing highest carbon. Therefore the values given have only a general significance, and individual tests may vary widely above or below the figures in the table.

For Bessemer or open-hearth acid process steel the tensile strength will ordinarily be greater for the same percentage of carbon given in this table, for the reason that the proportions of phosphorus and sulphur, and sometimes manganese, are usually higher than in open-hearth basic steel, each of these elements contributing to strength and hardness in the steel.

For convenient distinguishing terms, it is customary to classify steel in three grades, "mild or soft," "medium," and "hard"; and although the different grades blend into each other, so that no line of distinction exists, in a general sense the grades below .15 carbon may be considered as "soft" steel, from .15 to .30 carbon as "medium," and above that "hard" steel. Each grade has its own advantages for the particular purpose to which it is adapted. The soft steel is well adapted for boiler-plate and similar uses, where its high ductility is advantageous. The medium grades are used for general structural purposes, while harder steel is especially adapted for axles and shafts, and any service where good wearing surfaces are desired. Mild steel has superior welding property as compared to hard steel, and will endure higher heat without injury. Steel below .10 carbon should be capable of doubling flat without fracture, after being chilled from a red heat

TABLE II.—OPEN-HEARTH BASIC STEEL.

Percentage of carbon.	Tensile strength in pounds per square inch.		Ductility.	
	Ultimate strength.	Elastic limit.	Stretch in 8 inches.	Reduction of fractured area.
.08	54,000	32,500	32 per cent.	60 per cent.
.09	54,800	33,000	31 "	58 "
.10	55,700	33,500	31 "	57 "
.11	56,500	34,000	30 "	56 "
.12	57,400	34,500	30 "	55 "
.13	58,200	35,000	29 "	54 "
.14	59,100	35,500	29 "	53 "
.15	60,000	36,000	28 "	52 "
.16	60,800	36,500	28 "	51 "
.17	61,600	37,000	27 "	50 "
.18	62,500	37,500	27 "	49 "
.19	63,300	38,000	26 "	48 "
.20	64,200	38,500	26 "	47 "
.21	65,000	39,000	25 "	46 "
.22	65,800	39,500	25 "	45 "
.23	66,600	40,000	24 "	44 "
.24	67,400	40,500	24 "	43 "
.25	68,200	41,000	23 "	42 "

in cold water. Steel of .15 carbon will occasionally submit to the same treatment, but will usually bend around a curve whose radius is equal to the thickness of the specimen; about 90 per cent. of specimens stand the latter bending test without fracture. As the steel becomes harder, its ability to endure this bending test becomes more exceptional, and when the carbon ratio becomes .20, little over twenty-five per cent. of specimens will stand the last-described bending test. Steel having about .40 per cent. carbon will usually harden sufficiently to cut soft iron and maintain an edge.

Elasticity of Steel.

As the material elongates or shortens under stress, the change of length is directly proportionate to the stress, and the material recovers its original length after removal of the stress, until the elastic limit is reached, when changes of length are no longer regu-

lar, and permanent set takes place, or the destruction of the material has begun.

In good material the stress at elastic limit, for either tension or compression, is usually about six-tenths of the ultimate tenacity.

The ductility under tensile strength is usually measured by the total elongation in a given length, or by the percentage of reduction of the fractured area, or by both.

The elasticity is measured by the change of length under stress below the elastic limit of the material. The elasticity of the various grades of steel are practically uniform, that is, each material will exhibit a uniform change of length under uniform stress below the elastic limit; but, as the elastic limit of the higher grades is greater than that of the lower or softer grades, the former will elongate or shorten to a greater extent than the latter before its elasticity is injured. This property is expressed by a modulus, which for either material will average about 29,000,000 lbs. That is, if the change of length could be extended sufficiently, it would require 29,000,000 lbs. per square inch of section to double the original length under tensile strain, or to shorten the length one-half under compression. Therefore, steel will extend or shorten $\frac{1}{29000000}$ part of its normal length for every pound per sectional inch in change of load.

Expansion by Heat.

Soft steel or iron will extend about $\frac{1}{150000}$ part of its length for each degree F. of elevation of temperature. For a variation in temperature of 100 degrees F., the change in length will be about one inch in 125 feet.

Weight or Specific Gravity of Steel.

The specific gravity of steel varies according to the purity of the metal, and also according to the degree of condensation imparted by the process of rolling or forging.

As a rule, mild steel has a higher specific gravity than hard steel, and both are lower than perfectly pure iron, but about two per cent. higher than ordinary commercial iron. Structural steel in comparatively small sections, having the composition denoted in the previous table of tensile strength, has the following specific gravity, corresponding to given carbon ratio:

Carbon, per cent.	Specific gravity.	Weight per cubic foot in pounds.
.10	7.860	489.92
.20	7.858	489.80
.30	7.856	489.67

In the form of rolled beams and largest commercial sections the weight will be slightly less than this.

The weights for steel sections given in this book are all calculated on a basis of 489.6 lbs. per cubic foot, or the sectional area in square inches multiplied by 3.4 equals the weight in pounds per foot.

Working Strength of Steel.

In designing steel roof trusses engineers generally allow about 16,000 lbs. per square inch for the working tensile strength of steel shapes, such as angles or channels, and about 18,000 lbs. for round or flat bars, when the quality of the material is to be tested, and it is known that the work will be first-class. For wind bracing a stress of 20,000 lbs. is often used. (See page 268 of Freitag's "Architectural Engineering.")

Where the material is not to be tested, the author would not recommend the use of greater unit strains than 14,000 lbs. for shapes and 15,000 lbs. for bars. For truss-rods obtained of an ordinary blacksmith, and which have perhaps been welded, not over 12,500 lbs. should be used.

The New York and Chicago building laws fix the limit of tensile stress in steel at 16,000 lbs.; the Boston law at 15,000 lbs.

MANUFACTURERS' SPECIFICATIONS GOVERNING THE PHYSICAL PROPERTIES OF STRUCTURAL STEEL.

Revised Oct. 23, 1896.

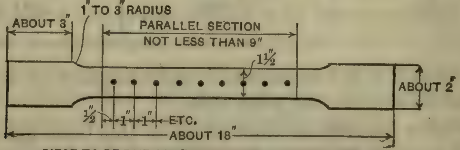
PROCESS OF MANUFACTURE.

(1) Steel may be made by either the open-hearth or Bessemer process.

TEST-PIECES.

(2) All tests and inspections shall be made at place of manufacture prior to shipment.

(3) The tensile strength, limit of elasticity, and ductility shall be determined from a standard test-piece cut from the finished material. The standard shape of the test-piece for sheared plates shall be as shown by the following sketch:



PIECE TO BE THE SAME THICKNESS AS THE PLATE

On tests cut from other material the test-piece may be either the same as for plates, or it may be planed or turned parallel throughout its entire length. The elongation shall be measured on an original length of 8 inches, except when the thickness of the finished material is $\frac{5}{16}$ inch or less, in which case the elongation shall be measured in a length equal to sixteen times the thickness; and except in rounds of $\frac{5}{8}$ inch or less in diameter, in which case the elongation shall be measured in a length equal to eight times the diameter of section tested. Two test-pieces shall be taken from each melt or blow of finished material, one for tension and one for bending.

ANNEALED TEST PIECES.

(4) Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

MARKING.

(5) Every finished piece of steel shall be stamped with the blow or melt number, and steel for pins shall have the blow or melt number stamped on the ends. Rivet and lacing steel, and small pieces for pin plates and stiffeners, may be shipped in bundles securely wired together, with the blow or melt number on a metal tag attached.

FINISH.

(6) Finished bars must be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

CHEMICAL PROPERTIES.

(7) Steel for buildings, train sheds, highway bridges and similar structures shall not contain more than 0.10 per cent. of phosphorus. Steel for railway bridges shall not contain more than 0.08 per cent. of phosphorus.

GRADES OF STEEL.

(8) Structural steel shall be of three grades: RIVET, SOFT, and MEDIUM.

RIVET STEEL.

(9) Ultimate strength, 48,000 to 58,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 26 per cent.

Bending test, 180 degrees flat on itself, without fracture on outside of bent portion.

SOFT STEEL.

(10) Ultimate strength, 52,000 to 62,000 pounds per square inch.

Elastic limit not less than one-half the ultimate strength.

Elongation, 25 per cent.

Bending test, 180 degrees flat on itself, without fracture on outside of bent portion.

MEDIUM STEEL

(11) Ultimate strength, 60,000 to 70,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 22 per cent.

Bending test, 180 degrees to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

PIN STEEL.

(12) Pins made from either of the above-mentioned grades of steel shall, on specimen test-pieces cut at a depth of one inch from surface of finished material, fill the physical requirements of the grade of steel from which it is rolled for ultimate strength, elastic limit, and bending, but the required elongation shall be decreased 5 per cent.

EYE-BAR STEEL.

(13) Eye-bar material, $1\frac{1}{2}$ inches and less in thickness, made of either of the above-mentioned grades of steel, shall, on test-pieces cut from finished material, fill the requirements of the grade of steel from which it is rolled. For thicknesses greater than $1\frac{1}{2}$ inches there will be allowed a reduction in the percentage of elongation of 1 per cent. for each $\frac{1}{8}$ of an inch increase of thickness, to a minimum of 20 per cent. for medium steel and 22 per cent. for soft steel.

FULL-SIZE TEST OF STEEL EYE-BARS.

(14) Full-size test of steel eye-bars shall be required to show not less than 10 per cent. elongation in the body of the bar, and tensile strength not more than 5,000 pounds below the minimum tensile strength required in specimen tests of the grade of steel from which they are rolled. The bars will be required to break in the body, but should a bar break in the head, but develop 10 per cent. elongation and the ultimate strength specified it shall not be cause for rejection, provided not more than one-third of the total number of bars tested break in the head; otherwise the entire lot will be rejected.

VARIATION IN WEIGHT.

(15) The variation in cross-section of weight of more than $2\frac{1}{2}$ per cent. from that specified will be sufficient cause for rejection, except in the case of sheared plates which will be covered by the following permissible variations:

a. Plates $12\frac{1}{2}$ pounds per square foot, or heavier, when ordered to weight, shall not average more than $2\frac{1}{2}$ per cent. variation above, or $2\frac{1}{2}$ per cent. below the theoretical weight.

b. Plates under $12\frac{1}{2}$ pounds per square foot, when ordered to weight, shall not average a greater variation than the following:

Up to 75 inches wide, $2\frac{1}{2}$ per cent. above, or $2\frac{1}{2}$ per cent. below the theoretical weight.

75 inches and over, 5 per cent. above, or 5 per cent. below the theoretical weight.

c. For all plates ordered to gauge, there will be permitted an average excess of weight over that corresponding to the dimensions on the order equal in amount to that specified in the following table.

TABLE OF ALLOWANCES FOR OVERWEIGHT FOR RECTANGULAR PLATES WHEN ORDERED TO GAUGE.

THE WEIGHT OF ONE CUBIC INCH OF ROLLED STEEL IS ASSUMED TO BE .2833 POUND.

(Plates ¼″ and over in thickness.)

Thickness of plate.	Width of plate.		
	Up to 57 inches.	75 in. to 100 in.	Over 100 inches.
¼ inch	10 per cent.	14 per cent.	18 per cent.
5/16 "	8 "	12 "	16 "
⅜ "	7 "	10 "	13 "
7/16 "	6 "	8 "	10 "
½ "	5 "	7 "	9 "
9/16 "	4½ "	6½ "	8½ "
⅝ "	4 "	6 "	8 "
Over ⅝ "	3½ "	5 "	6½ "

For Ordinary Building Construction the following form of specification for the *quality and testing* of the steel work is recommended:

Specifications for Structural Steel Work.

Material and Workmanship.—The entire structural framework as indicated by the framing plans, or as specified, is to be of wrought steel, of quality hereinafter designated; all material to be provided and put in place by this contractor unless specifically stated to the contrary. All work to be done in a neat and skilful manner, as per detail or specified, and if not detailed or specified, as directed by the superintendent.

Quality and Material.—Steel may be made by either the Bessemer or open-hearth process, but must be uniform in quality, and in no case contain over $\frac{1}{10}$ of one per cent. of phosphorus.

The grade of steel used (except for rivets) shall fill the following requirements when tested in small specimens:

[Here should be inserted section (11) of the foregoing specifications.]

Inspection.—All steel work is to be inspected from the melt to final delivery of finished material on board cars. The inspection

will include surface, mill, and shop inspection by an inspector satisfactory to the architect or his engineer, to whom all reports are to be made. No work shall be delivered until approved and stamped by the inspector. All inspection shall be at the expense of this contractor.

Tests.—[Sections (3) and (4) in preceding specification to be inserted here.]

Eye-bars.—To determine the strength of the eyes two full-size eye-bars with eyes shall be tested to destruction. These tests shall show—[Section 14 in preceding specification to follow.]

Finish.—Finished bars must be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

Rivet Steel.—[Same as section (9), preceding specification.]

Rivets.—The pitch of rivets shall never be less than $1\frac{1}{2}''$ nor more than $6''$, while the minimum distance from the centre of any rivet to the edge of the shape shall be $1\frac{1}{4}''$. No rivets to be used in tension. An excess of 25 per cent. shall be allowed in proportioning field rivets.

Rivet-holes may be punched or drilled, but must not be more than $\frac{1}{16}''$ larger than diameter of rivet.

Rivet-holes must be accurately spaced, as drift-pins will be allowed for assembling only.

The rivets shall completely fill the holes, with full heads concentric with the rivets, and in full contact with the surface of the metal.

Tie-bars, Eye-bars, Screw Ends, Clevises, Sleeve-nuts, and Turn-buckles.

The best shape for an iron or steel tie is largely determined by the manner in which the tie is to be secured at the ends. If the tie is to be secured by rivets, either channels or angles are generally used, except where only a very small bar is required, in which case a plain rectangular bar may be used. In figuring the strength of such ties, it is customary to use the net sectional area of the tie at the point where the area is most reduced by rivet-holes.

For figuring the reduction in sectional area by rivet- or bolt-holes, Table XI. of this chapter will be found very convenient.

Eye-bars.—For pin-connected trusses, the ties almost invariably consist of eye-bars, i.e., a rectangular bar with an eye at each end. "Eye-bars are now generally made of mild steel, of

an ultimate strength of from 56,000 to 66,000 lbs. per square inch, the methods of manufacture securing a more satisfactory and reliable product from that metal than from iron. Steel eye-bars are made by forging or upsetting the eye or head of the bar in a die, and subsequently reheating and annealing the finished bars previous to boring the pin-holes. Wrought-iron bars are made by piling and welding, which is always an unreliable process.

For economy in dies, the same head or eye is used for two or three different pin-holes, and for this reason it is often cheaper to use a slightly larger head than would be really necessary for strength, rather than to have a special die made to order. Table V. gives the principal dimensions of the standard sizes of steel eye-bars manufactured by the Edge Moor Bridge Works. Eye-bars made by other companies vary slightly from those dimensions and from each other, but not to any great extent. The thickness of the bar for any given width should not be less than the minimum thickness given in the table, because thinner bars are difficult to manufacture, and are liable to buckle in the head when under strain. "The thickness of the bar may be made anything greater than this minimum, but a thickness of two inches for bars six inches wide and under is rarely exceeded." The thicker the bar the greater will be the bending moment on the pin.

"It is always better to use an eye the diameter of which is about two and one-quarter times the width of the bar. In extreme cases the diameter of the eye may be made two and one-half times the width of the bar, but it is never desirable to exceed this, as the cost and difficulty of manufacture increase rapidly if larger eyes are used. Eye-bars are now made as large as 12×3 inches, with eyes 27 to 30 inches in diameter."*

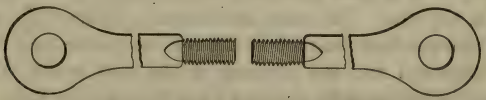


Fig. 1.—Eye-bar with screw ends for sleeve-nut or turn-buckle.

Eye-bars are sometimes made with upset screw ends and sleeve-nuts, or turn-buckles in the centre, as shown by Fig. 1.

* C. W. Bryan, C.E., Engineer of the Edge Moor Bridge Works.

Light square rods, secured to large pins, are often made with loop eyes, as shown by Figs. 2 and 3, as for such eyes the diameter of the pin is not limited.

Loop eyes are made by welding, and as satisfactory welds can-

Fig. 2.—Loop-eyes and sleeve-nuts.

not be generally secured with steel, loop-ended rods are usually made of wrought iron.

When two single tie-rods balance each other on a pin, to avoid eccentricity one of the rods must either have a clevis on the end,

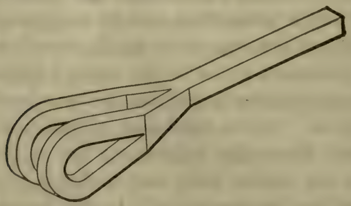


Fig. 3.—Forked loop.

as shown at the head of Table VI., or a forked loop, as in Fig. 3. Clevises also afford means of adjusting the length of the tie.

Sleeve-nuts and Turn-buckles.—For adjusting the length of the tie-bars or rods, which pass over a pin, sleeve-nuts or turn-buckles are used, and even when the end of the rod is held by a nut, as in wooden trusses, it is often desirable to place the turn-buckle in the centre of the rod, for adjusting after the truss has seasoned, as it is then generally inconvenient if not impossible to get at the nut. The open turn-buckle, Table VII., possesses the advantages that the ends of the rod are visible, and it may be easily inspected and the position of the rods noted; also, that they may be adjusted by running a bar through the link. Tables VII. and VIII. give dimensions of sleeve-nuts and turn-buckles which, while not the same with all manufacturers, are very nearly so.

Upset Screw Ends.—When a screw thread is cut on a rod or bolt, the strength of the rod or bolt is measured by the sectional

area at the root of the thread, and consequently there is a considerable excess of metal in the body of the rod that is practically wasted. For long rods, therefore, and especially where there are many of a kind, the end of the rod is enlarged or upset by forging, so that, when the screw is cut, the diameter of the screw at the root of the thread is left *a little larger than the body of the rod.* Frequent trials with such rods have proven that they will pull apart in tension anywhere else but in the screw; the threads remaining perfect, and the nut turning freely after having been subjected to such a severe test. By this means the net section required in tension is made available with the least excess of material, and no more dead weight is put upon the structure than is actually needed to carry the loads imposed.

Only the larger machine shops, however, are equipped for upsetting, so that in small towns and cities it is often necessary to send to a considerable distance for upset rods. For this reason it is often cheaper to use a slightly larger rod without upsetting, than to specify the theoretical size with upset ends. Upset rods also require a larger hole to pass through.

Dimensions of upset screw ends are given in Table IV.

Tables.—The following tables will be found useful when designing ties of steel or iron, or for drawing turn-buckles, sleeve-nuts, clevises, etc. The strength of the plain rods in Table III. are based on the sectional area at the root of the thread.

TABLE III.—STRENGTH OF ROUND RODS OF IRON AND STEEL.

Diameter in inches.	Not Upset. Allowed strain per sq. in.*			Upset. Allowed strain per sq. in.*		
	10,000 lbs.	12,500 lbs.	15,000 lbs.	10,000 lbs.	12,500 lbs.	15,000 lbs.
1/4	268	335	402	491	613	736
5/16	452	565	678	767	958	1150
3/8	679	848	1,018	1,104	1,380	1,656
7/16	929	1,160	1,393	1,503	1,878	2,254
1/2	1,256	1,570	1,884	1,963	2,453	2,944
9/16	1,618	2,022	2,427	2,485	3,106	3,727
5/8	1,963	2,453	2,944	3,068	3,835	4,600
3/4	3,000	3,750	4,500	4,418	5,520	6,627
7/8	4,200	5,250	6,300	6,013	7,516	9,020
1	5,430	6,780	8,140	7,854	9,815	11,780
1 1/8	6,860	8,570	10,290	9,940	12,425	14,900
1 1/4	8,850	11,060	13,270	12,270	15,330	18,400
1 3/8	10,700	13,370	16,050	14,840	18,550	22,260
1 1/2	12,870	16,080	19,300	17,670	22,080	26,500
1 5/8	15,000	18,750	22,500	20,730	25,910	31,090
1 3/4	17,600	23,000	26,400	24,050	30,060	36,070
1 7/8	20,200	25,250	30,300	27,610	34,500	41,400
2	22,800	28,500	34,200	31,420	39,270	47,130
2 1/8	26,400	33,000	39,600	35,460	44,320	53,190
2 1/4	30,000	37,500	45,000	39,760	49,700	59,680
2 3/8	33,500	41,870	50,250	44,300	55,370	66,450
2 1/2	37,200	46,500	55,800	49,080	61,350	73,620
2 3/4	46,400	58,000	69,800	59,390	74,230	89,080
3	54,000	67,500	81,000	70,680	88,350	106,000
3 1/4	65,000	81,250	97,500	82,950	103,690	124,400
3 1/2	75,400	94,250	113,100	96,210	120,260	144,300
3 3/4	85,600	107,000	128,400	110,450	138,060	165,600
4	99,000	123,750	148,500	125,660	157,000	188,490
4 1/4	113,400	141,700	170,100	141,800	177,250	212,700
4 1/2	126,000	157,500	189,000	159,000	198,750	238,500
4 3/4	141,800	177,250	212,700	177,200	221,500	265,800
5	157,600	197,000	236,400	196,300	245,370	298,400
5 1/4	175,900	219,870	263,850	216,400	270,500	324,000
5 1/2	192,600	240,750	288,900	237,500	296,800	356,000
5 3/4	212,300	265,370	318,400	259,600	324,500	389,000
6	231,000	288,750	346,500	282,700	353,300	424,000

* For first-class work and material 12,500 lbs. may be allowed for iron and 15,000 lbs. for steel. If the rods are to be welded or are made by an ordinary blacksmith use 10,000 lbs. for iron and 12,500 lbs. for steel.

TABLE IV.—STANDARD PROPORTIONS OF UPSET SCREW-ENDS FOR ROUND AND SQUARE BARS.

Diameter of round or side of square bar.	Round Bars.				Square Bars.			
	Diameter of upset screw-end.	Diam. of screw at root of thread.	Threads per inch.	Excess of effective area of screw-end over bar.	Diameter of upset screw-end.	Diam. of screw at root of thread.	Threads per inch.	Excess of effective area of screw-end over bar.
Inches	Inches	Inches.	No.	Per cent	Inches	Inches.	No.	Per cent
1/2	3/4	0.620	10	54	3/4	0.620	10	21
9/16	3/4	0.620	10	21	7/8	0.731	9	33
5/8	7/8	0.731	9	37	1	0.837	8	41
11/16	1	0.837	8	48	1	0.837	8	17
3/4	1	0.837	8	25	1 1/8	0.940	7	23
13/16	1 1/8	0.940	7	34	1 1/4	1.065	7	35
7/8	1 1/4	1.065	7	48	1 3/8	1.160	6	38
15/16	1 1/4	1.065	7	29	1 3/8	1.160	6	20
1	1 3/8	1.160	6	35	1 1/2	1.284	6	29
1 1/16	1 3/8	1.160	6	19	1 5/8	1.389	5 1/2	34
1 1/8	1 1/2	1.284	6	30	1 5/8	1.389	5 1/2	20
1 3/16	1 1/2	1.284	6	17	1 3/4	1.490	5	24
1 1/4	1 5/8	1.389	5 1/2	23	1 7/8	1.615	5	31
1 5/16	1 3/4	1.490	5	29	1 7/8	1.615	5	19
1 3/8	1 3/4	1.490	5	18	2	1.712	4 1/2	22
1 7/16	1 7/8	1.615	5	26	2 1/8	1.837	4 1/2	28
1 1/2	2	1.712	4 1/2	30	2 1/8	1.837	4 1/2	18
1 9/16	2	1.712	4 1/2	20	2 1/4	1.962	4 1/2	24
1 5/8	2 1/8	1.837	4 1/2	28	2 3/8	2.087	4 1/2	30
1 11/16	2 1/8	1.837	4 1/2	18	2 3/8	2.087	4 1/2	20
1 3/4	2 1/4	1.962	4 1/2	26	2 1/2	2.175	4	21
1 13/16	2 1/4	1.962	4 1/2	17	2 5/8	2.300	4	26
1 7/8	2 3/8	2.087	4 1/2	24	2 5/8	2.300	4	18
1 15/16	2 1/2	2.175	4	26	2 3/4	2.425	4	23
2	2 1/2	2.175	4	18	2 7/8	2.550	4	28
2 1/16	2 5/8	2.300	4	24	2 7/8	2.550	4	20
2 1/8	2 5/8	2.300	4	17	3	2.629	3 1/2	20
2 3/16	2 3/4	2.425	4	23	3 1/8	2.754	3 1/2	24

TABLE IV.—UPSET SCREW-ENDS—(*concluded*).

Diameter of round or side of square bar.	Round Bars. Diameter of upset screw-end.	Round Bars. Diam. of screw at root of thread.	Round Bars. Threads per inch.	Round Bars. Excess of effective area of screw-end over bar.	Square Bars. Diameter of upset screw-end	Square Bars. Diam. of screw at root of thread.	Square Bars. Threads per inch.	Square Bars. Excess of effective area of screw-end over bar.
Inches	Inches	Inches	No.	Per cent	Inches	Inches	No.	Per cent
2¼	2⅞	2.550	4	28	3⅛	2.754	3½	18
2 5/16	2⅞	2.550	4	22	3¼	2.879	3½	22
2⅜	3	2.629	3½	23	3⅜	3.004	3½	26
2 7/16	3⅛	2.754	3½	28	3⅜	3.004	3½	19
2½	3⅛	2.754	3½	21	3½	3.100	3¼	21
2 9/16	3¼	2.879	3½	26	3⅝	3.225	3¼	24
2⅝	3¼	2.879	3½	20	3⅝	3.225	3¼	19
2 11/16	3⅜	3.004	3½	25	3¾	3.317	3	20
2¾	3⅜	3.004	3½	19	3⅞	3.442	3	23
2 13/16	3½	3.100	3¼	22	3⅞	3.442	3	18
2⅞	3⅝	3.225	3¼	26	4	3.567	3	21
2 15/16	3⅝	3.225	3¼	21	4⅛	3.692	3	24
3	3¾	3.317	3	22	4⅛	3.692	3	19
3⅛	3⅞	3.442	3	21	4⅜	3.923	2⅞	24
3¼	4	3.567	3	20	4½	4.028	2¾	21
3⅜	4⅛	3.692	3	20	4⅝	4.153	2¾	19
3½	4¼	3.798	2⅞	18				
3⅝	4½	4.028	2¾	23				
3¾	4⅝	4.153	2¾	23				
3⅞	4¾	4.255	2⅝	21				

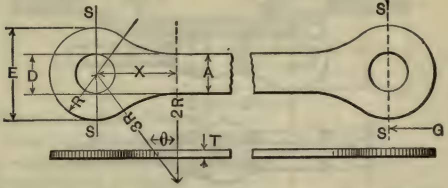
REMARKS.—As upsetting reduces the strength of iron, bars having the same diameter at root of thread as that of the bar invariably break in the screw-end, when tested to destruction, without developing the full strength of the bar. It is therefore necessary to make up for this loss in strength by an excess of metal in the upset screw-ends over that in the bar.

The above table is the result of numerous tests on finished bars made at the Keystone Bridge Company's Works in Pittsburgh, and gives proportions that will cause the bar to break in the body in preference to the upset end.

The screw-threads in above table are the Franklin Institute standard.

To make one upset end for five inches length of thread allow six inches length of rod additional.

TABLE V.—STEEL EYE-BARS.
Edge Moor Bridge Works' Standard.

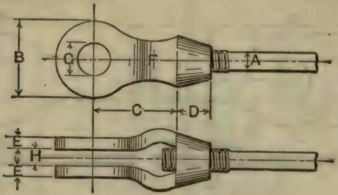


A Width of body of bar.	T Minimum thickness of bar.	E Diameter of head of bar.	D Diameter of largest pin-hole.	Sectional area of the head on line S–S in excess of that in body of bar.	Safe strength at 15,000 lbs. per sq. in.
Inches.	Inches.	Inches.	Inches.	Per cent.	Pounds.
2	½	4½	1⅞	33	15,000
2	½	5½	2⅞	33	15,000
2½	⅝	5½	2⅛	33	23,430
2½	⅝	6½	3⅛	33	23,430
3	¾	6½	2½	33	33,750
3	¾	8	4	33	33,750
3	¾	9	5	33	33,750
4	¾	9½	4⅛	33	45,000
4	¾	10½	5⅛	33	45,000
4	¾	11½	6⅛	33	45,000
5	¾	11½	4⅝	37	56,250
5	¾	12½	5⅝	37	56,250
5	1	13	6⅛	37	75,000
5	1	14	7⅛	37	75,000
6	⅞	13½	5¼	37	78,750
6	⅞	14½	6¼	37	78,750
6	1	15½	7¼	37	90,000
7	15/16	15½	5⅝	40	98,400
7	15/16	17	7⅛	40	98,400
8	1	17	5¾	40	120,000
8	1	18	6¾	40	120,000
8	1	19	8	40	120,000
9	1⅛	19½	7	40	151,875
9	1⅛	21½	9	40	151,875
9	1¼	22½	10	..	168,750
10	1⅜	24½	10⅝	..	206,250

The size of head given is the size of die. The size of finished head will overrun this about ¼″. Eye-bars are hydraulic forged without the addition of extraneous metal and without buckles or welds. The heads on eye-bars are finished of the same thickness "T" as body of bar.

TABLE VI.—STANDARD CLEVIS NUTS.

The Carnegie Steel Company, Limited.

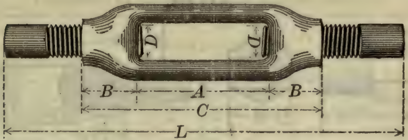


(Distance H can be made to suit connections.)

Diameter of round bar.	A Upset screw end for round bar.	Side of square bar.	A Upset screw end for square bar.	B Diameter of eye.	C L'gth of fork.	D L'gth of thread	E Thickness of bar in fork.	F Width of bar in fork.	G Diameter of pin.
$1\frac{1}{4}$*	$1\frac{5}{8}$	$1\frac{1}{8}$	$1\frac{5}{8}$	$4\frac{3}{4}$	$5\frac{1}{2}$	2	$\frac{5}{8}$	$2\frac{1}{2}$	$1\frac{7}{8}$
$1\frac{5}{16}$	$1\frac{3}{4}$	$1\frac{3}{16}$	$1\frac{3}{4}$	$5\frac{7}{8}$	$6\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}$	$3\frac{3}{16}$	$2\frac{1}{4}$
$1\frac{3}{8}$	$1\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{7}{8}$	$5\frac{7}{8}$	$6\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}$	$3\frac{3}{16}$	$2\frac{1}{4}$
$1\frac{7}{16}$	$1\frac{7}{8}$	$1\frac{5}{16}$	$1\frac{7}{8}$	$5\frac{7}{8}$	$6\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}$	$3\frac{3}{16}$	$2\frac{1}{4}$
$1\frac{1}{2}$	2	$1\frac{3}{8}$	2	$5\frac{7}{8}$	$6\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}$	$3\frac{3}{16}$	$2\frac{1}{4}$
$1\frac{9}{16}$	2	$1\frac{7}{16}$	$2\frac{1}{8}$	$5\frac{7}{8}$	$6\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}$	$3\frac{3}{16}$	$2\frac{1}{4}$
$1\frac{5}{8}$	$2\frac{1}{8}$	.....	.....	$5\frac{7}{8}$	$6\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}$	$3\frac{3}{16}$	$2\frac{1}{4}$
$1\frac{11}{16}$	$2\frac{1}{8}$	$1\frac{1}{2}$	$2\frac{1}{8}$	$6\frac{5}{8}$	7	$2\frac{7}{8}$	$\frac{7}{8}$	$3\frac{9}{16}$	$2\frac{5}{8}$
$1\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{9}{16}$	$2\frac{1}{4}$	$6\frac{5}{8}$	7	$2\frac{7}{8}$	$\frac{7}{8}$	$3\frac{9}{16}$	$2\frac{5}{8}$
$1\frac{13}{16}$	$2\frac{1}{4}$	$1\frac{5}{8}$	$2\frac{3}{8}$	$6\frac{5}{8}$	7	$2\frac{7}{8}$	$\frac{7}{8}$	$3\frac{9}{16}$	$2\frac{5}{8}$
$1\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{11}{16}$	$2\frac{3}{8}$	$6\frac{5}{8}$	7	$2\frac{7}{8}$	$\frac{7}{8}$	$3\frac{9}{16}$	$2\frac{5}{8}$
$1\frac{15}{16}$	$2\frac{1}{2}$	$1\frac{3}{4}$	$2\frac{1}{2}$	$7\frac{7}{8}$	8	$3\frac{1}{2}$	$1\frac{1}{8}$	$3\frac{7}{8}$	$3\frac{1}{8}$
2	$2\frac{1}{2}$	$1\frac{13}{16}$	$2\frac{5}{8}$	$7\frac{7}{8}$	8	$3\frac{1}{2}$	$1\frac{1}{8}$	$3\frac{7}{8}$	$3\frac{1}{8}$
$2\frac{1}{16}$	$2\frac{5}{8}$	$1\frac{7}{8}$	$2\frac{5}{8}$	$7\frac{7}{8}$	8	$3\frac{1}{2}$	$1\frac{1}{8}$	$3\frac{7}{8}$	$3\frac{1}{8}$
$2\frac{1}{8}$	$2\frac{5}{8}$	$1\frac{15}{16}$	$2\frac{3}{4}$	$7\frac{7}{8}$	8	$3\frac{1}{2}$	$1\frac{1}{8}$	$3\frac{7}{8}$	$3\frac{1}{8}$
$2\frac{3}{16}$	$2\frac{3}{4}$	2	$2\frac{7}{8}$	$7\frac{7}{8}$	8	$3\frac{1}{2}$	$1\frac{1}{8}$	$3\frac{7}{8}$	$3\frac{1}{8}$
$2\frac{1}{4}$	$2\frac{7}{8}$	.....	.....	$7\frac{7}{8}$	8	$3\frac{1}{2}$	$1\frac{1}{8}$	$3\frac{7}{8}$	$3\frac{1}{8}$
$2\frac{5}{16}$	$2\frac{7}{8}$	$2\frac{1}{16}$	$2\frac{7}{8}$	9	$8\frac{1}{2}$	4	$1\frac{1}{4}$	$4\frac{13}{16}$	$3\frac{5}{8}$
$2\frac{3}{8}$	3	$2\frac{1}{8}$	3	9	$8\frac{1}{2}$	4	$1\frac{1}{4}$	$4\frac{13}{16}$	$3\frac{5}{8}$
$2\frac{7}{16}$	$3\frac{1}{8}$	$2\frac{3}{16}$	$3\frac{1}{8}$	9	$8\frac{1}{2}$	4	$1\frac{1}{4}$	$4\frac{13}{16}$	$3\frac{5}{8}$
$2\frac{1}{2}$	$3\frac{1}{8}$	$2\frac{1}{4}$	$3\frac{1}{8}$	9	$8\frac{1}{2}$	4	$1\frac{1}{4}$	$4\frac{13}{16}$	$3\frac{5}{8}$
$2\frac{9}{16}$	$3\frac{1}{4}$	$2\frac{5}{16}$	$3\frac{1}{4}$	9	$8\frac{1}{2}$	4	$1\frac{1}{4}$	$4\frac{13}{16}$	$3\frac{5}{8}$
$2\frac{5}{8}$	$3\frac{1}{4}$	$2\frac{3}{8}$	$3\frac{3}{8}$	9	$8\frac{1}{2}$	4	$1\frac{1}{4}$	$4\frac{13}{16}$	$3\frac{5}{8}$
$2\frac{11}{16}$	$3\frac{3}{8}$	.....	.....	9	$8\frac{1}{2}$	4	$1\frac{1}{4}$	$4\frac{13}{16}$	$3\frac{5}{8}$
$2\frac{3}{4}$	$3\frac{3}{8}$	$2\frac{7}{16}$	$3\frac{3}{8}$	$9\frac{3}{4}$	9	$4\frac{1}{4}$	$1\frac{5}{8}$	$5\frac{1}{4}$	$3\frac{7}{8}$
$2\frac{13}{16}$	$3\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{1}{2}$	$9\frac{3}{4}$	9	$4\frac{1}{4}$	$1\frac{5}{8}$	$5\frac{1}{4}$	$3\frac{7}{8}$
$2\frac{7}{8}$	$3\frac{5}{8}$	$2\frac{9}{16}$	$3\frac{5}{8}$	$9\frac{3}{4}$	9	$4\frac{1}{4}$	$1\frac{5}{8}$	$5\frac{1}{4}$	$3\frac{7}{8}$
$2\frac{15}{16}$	$3\frac{5}{8}$	$2\frac{5}{8}$	$3\frac{5}{8}$	$9\frac{3}{4}$	9	$4\frac{1}{4}$	$1\frac{5}{8}$	$5\frac{1}{4}$	$3\frac{7}{8}$

* This clevis used for all smaller bars.

TABLE VII.—TURNBUCKLES.



D. Size = diameter of screw.
A. Length in clear between heads.
B. Length of tapped heads = $1\frac{1}{2}$D.
C. Total length of buckle.
L. Total length of buckle and stub ends when open.

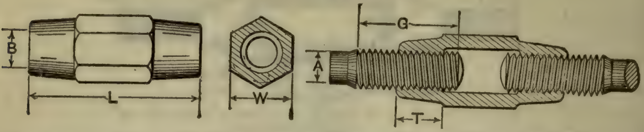
Size D	A	B	C	L	Size D	A	B	C	L
$\frac{3}{8}$	6	$\frac{9}{16}$	$7\frac{1}{8}$	22	$1\frac{3}{4}$	6	$2\frac{5}{8}$	$11\frac{1}{4}$	28
$\frac{7}{16}$	6	$\frac{21}{32}$	$7\frac{5}{16}$	22	$1\frac{7}{8}$	6	$2\frac{13}{16}$	$11\frac{5}{8}$	29
$\frac{1}{2}$	6	$\frac{3}{4}$	$7\frac{1}{2}$	22	2	6	3	12	29
$\frac{9}{16}$	6	$\frac{27}{32}$	$7\frac{11}{16}$	22	$2\frac{1}{8}$	6	$3\frac{5}{16}$	$12\frac{3}{8}$	29
$\frac{5}{8}$	6	$\frac{15}{16}$	$7\frac{7}{8}$	22	$2\frac{1}{4}$	6	$3\frac{3}{8}$	$12\frac{3}{4}$	30
$\frac{3}{4}$	6	$1\frac{1}{8}$	$8\frac{1}{4}$	23	$2\frac{3}{8}$	6	$3\frac{9}{16}$	$13\frac{1}{8}$	31
$\frac{7}{8}$	6	$1\frac{5}{16}$	$8\frac{5}{8}$	24	$2\frac{1}{2}$	6	$3\frac{3}{4}$	$13\frac{1}{2}$	32
1	6	$1\frac{1}{2}$	9	25	$2\frac{5}{8}$	6	$3\frac{15}{16}$	$13\frac{7}{8}$	32
$1\frac{1}{8}$	6	$1\frac{11}{16}$	$9\frac{3}{8}$	25	$2\frac{3}{4}$	6	$4\frac{1}{8}$	$14\frac{1}{4}$	33
$1\frac{1}{4}$	6	$1\frac{7}{8}$	$9\frac{3}{4}$	26	$2\frac{7}{8}$	6	$4\frac{5}{16}$	$14\frac{5}{8}$	33
$1\frac{3}{8}$	6	$2\frac{1}{16}$	$10\frac{1}{8}$	27	3	6	$4\frac{1}{2}$	15	34
$1\frac{1}{2}$	6	$2\frac{1}{4}$	$10\frac{1}{2}$	27	$3\frac{1}{4}$	6	$4\frac{7}{8}$	$15\frac{3}{4}$	36
$1\frac{5}{8}$	6	$2\frac{7}{16}$	$10\frac{7}{8}$	28	$3\frac{1}{2}$	6	$5\frac{1}{4}$	$16\frac{1}{2}$	37

Lengths given above are standard for bridge, roof, and ordinary truss buckles.

They have a guaranteed strength of 60,000 pounds per square inch of section of bolt at bottom of thread. Stub bolt ends are made of good bridge iron having tensile strength of 50,000 pounds per square inch.

Open buckles of this form can be adjusted with a bar, hook, or wrench, and have the great advantage of showing the ends of the bolts, so that inspectors can see that they have a good hold of thread and do not butt together.

TABLE VIII.—RIGHT AND LEFT NUTS OR SLEEVE-NUTS.



DIMENSIONS OF NUTS FROM EDGE MOOR BRIDGE WORKS' STANDARD.

B Diam. of screw.	G L'gth of upset.	A Diameter of bar.	A Side of square bar.	L Length of nut.	T L'gth of thr'd.	W Dia. of hex.	Weight of One nut.	Weight of One nut and two screw-ends.
Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Lbs.	Lbs.
				ord. l'ths.				
7/8	4 1/2	5/8	9/16	6	1 7/16	1 5/8	1 3/4	4 1/4
1	4 1/2	11/16 and 3/4	5/8 and 11/16	6	1 7/16	1 5/8	1 3/4	4 1/4
1 1/8	4 3/4	13/16	3/4	6 1/2	1 5/8	2	3	7 1/2
1 1/4	4 3/4	7/8 " 15/16	13/16	6 1/2	1 5/8	2	3	7 1/2
1 3/8	5	1 " 1 1/16	7/8 " 15/16	7	1 7/8	2 3/8	4 3/4	11 3/4
1 1/2	5	1 1/8 " 1 3/16	1	7	1 7/8	2 3/8	4 3/4	11 3/4
1 5/8	5 1/4	1 1/4	1 1/16 " 1 1/8	7 1/2	2 1/16	2 3/4	6 3/4	16 3/4
1 3/4	5 1/4	1 5/16 " 1 3/8	1 3/16	7 1/2	2 1/16	2 3/4	6 3/4	16 3/4
1 7/8	5 1/2	1 7/16	1 1/4 " 1 5/16	8	2 5/16	3 1/8	9 1/4	23 1/4
2	5 1/2	1 1/2 " 1 9/16	1 3/8	8	2 5/16	3 1/8	9 1/4	23 1/4
2 1/8	5 3/4	1 5/8 " 1 11/16	1 7/16 " 1 1/2	8 1/2	2 1/2	3 1/2	12 1/2	31 1/2
2 1/4	5 3/4	1 3/4 " 1 13/16	1 9/16	8 1/2	2 1/2	3 1/2	12 1/2	31 1/2
2 3/8	6	1 7/8	1 5/8 " 1 11/16	9	2 3/4	3 7/8	16 3/4	41 3/4
2 1/2	6	1 15/16 " 2	1 3/4	9	2 3/4	3 7/8	16 3/4	41 3/4
2 5/8	6 1/4	2 1/16 " 2 1/8	1 13/16 " 1 7/8	9 1/2	2 15/16	4 1/4	21 1/2	53 1/4
2 3/4	6 1/4	2 3/16	1 15/16	9 1/2	2 15/16	4 1/4	21 1/2	53 1/4
2 7/8	6 1/2	2 1/4 " 2 5/16	2 " 2 1/16	10	3 3/16	4 5/8	26 1/2	66 1/4
3	6 1/2	2 3/8	2 1/8	10	3 3/16	4 5/8	26 1/2	66 1/4
3 1/4	6 3/4	2 9/16 " 2 5/8	2 5/16	10 1/2	3 3/8	5	32	81
3 1/2	7	2 13/16	2 1/2	11	3 5/8	5 3/8	38 1/4	97 3/4
3 3/4	7 1/4	3	2 11/16	11 1/2	3 13/16	5 3/4	45	116
4	7 1/2	3 1/4	2 7/8	12	4 1/16	6 1/8	53 1/2	138
				ext. l'ths.				
1 1/4	4 3/4	7/8 " 15/16	13/16	12	2 1/8	2		
1 1/8	4 3/4	13/16	3/4	8 1/2	1 5/8	2	4	9 3/4
1 1/4	4 3/4	7/8 " 15/16	13/16	8 1/2	1 5/8	2	4	9 3/4
1 3/8	5	1 " 1 1/16	7/8 " 15/16	9	1 7/8	2 3/8	6 1/4	15 1/4
1 1/2	5	1 1/8 " 1 3/16	1	9	1 7/8	2 3/8	6 1/4	15 1/4
1 5/8	5 1/4	1 1/4	1 1/16 " 1 1/8	9 1/2	2 1/16	2 3/4	8 3/4	21 1/2
1 3/4	5 1/4	1 5/16 " 1 3/8	1 3/16	9 1/2	2 1/16	2 3/4	8 3/4	21 1/2
1 7/8	5 1/2	1 7/16	1 1/4 " 1 5/16	10	2 5/16	3 1/8	12 1/4	29 3/4
2	5 1/2	1 1/2 " 1 9/16	1 3/8	10	2 5/16	3 1/8	12 1/4	29 3/4

Length of upset ends for use with right and left nuts may be made one inch shorter than the dimensions given in column "G" above.

TABLE IX.—SAFE STRENGTH OF FLAT ROLLED BARS.

(Computed at 10,000 lbs. per square inch.) *

Thickness in inches.	Width in inches.									
	1″	$1\frac{1}{4}$″	$1\frac{1}{2}$″	$1\frac{3}{4}$″	2″	$2\frac{1}{4}$″	$2\frac{1}{2}$″	$2\frac{3}{4}$″	3″	$3\frac{1}{4}$″
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
$\frac{1}{16}$	360	780	940	1,090	1,250	1,410	1,560	1,720	1,880	2,030
$\frac{1}{8}$	1,250	1,560	1,880	2,190	2,500	2,810	3,130	3,440	3,750	4,060
$\frac{3}{16}$	1,880	2,340	2,810	3,280	3,750	4,220	4,690	5,160	5,630	6,090
$\frac{1}{4}$	2,500	3,130	3,750	4,380	5,000	5,630	6,250	6,880	7,500	8,130
$\frac{5}{16}$	3,130	3,910	4,690	5,470	6,250	7,030	7,810	8,590	9,380	10,200
$\frac{3}{8}$	3,750	4,690	5,630	6,560	7,500	8,440	9,380	10,300	11,300	12,200
$\frac{7}{16}$	4,380	5,470	6,560	7,660	8,750	9,840	10,900	12,000	13,100	14,200
$\frac{1}{2}$	5,000	6,250	7,500	8,750	10,000	11,300	12,500	13,800	15,000	16,300
$\frac{9}{16}$	5,630	7,030	8,440	9,840	11,300	12,700	14,100	15,500	16,900	18,300
$\frac{5}{8}$	6,250	7,810	9,380	10,900	12,500	14,100	15,600	17,200	18,800	20,300
$\frac{11}{16}$	6,880	8,590	10,300	12,000	13.800	15,500	17,200	18,900	20,600	22,300
$\frac{3}{4}$	7,500	9,380	11,300	13,100	15,000	16,900	18,800	20,600	22,500	24,400
$\frac{13}{16}$	8,130	10,200	12,200	14,200	16,300	18,300	20,300	22,300	24,400	26,400
$\frac{7}{8}$	8,750	10,900	13,100	15,300	17,500	19,700	21,900	24,100	26,300	28,400
$\frac{15}{16}$	9,380	11,700	14,100	16,400	18,800	21,100	23,400	25,800	28,100	30,500
1	10,000	12,500	15,000	17,500	20,000	22,500	25,000	27,500	30,000	32,500
$1\frac{1}{16}$	10,600	13,300	15,900	18,600	21,300	23,900	26,600	29,200	31,900	34,500
$1\frac{1}{8}$	11,300	14,100	16,900	19,700	22,500	25,300	28,100	30,900	33,800	36,600
$1\frac{3}{16}$	11,900	14,800	17,800	20,800	23,800	26,700	29,700	32,700	35,600	38,600
$1\frac{1}{4}$	12,500	15,600	18,800	21,900	25,000	28,100	31,300	34,400	37,500	40,600
$1\frac{3}{8}$	13,800	17,200	20,600	24,100	27,500	30,900	34,400	37,800	41,300	44,700
$1\frac{1}{2}$	15,000	18,800	22,500	26,300	30,000	33,800	37,500	41,300	45,000	48,800
$1\frac{5}{8}$	16,300	20,300	24,400	28,400	32,500	36,600	40,600	44,700	48,800	52,800
$1\frac{3}{4}$	17,500	21,900	26,300	30,600	35,000	39,400	43,800	48,100	52,500	56,900
$1\frac{7}{8}$	18,800	23,400	28,100	32,800	37,500	42,200	46,900	51,600	56,300	60,900
2	20,000	25,000	30,000	35,000	40,000	45,000	50,000	55,000	60,000	65,000

* For unit stresses of 12,000, 12,500, and 15,000 lbs. increase by $\frac{1}{5}$, $\frac{1}{4}$, and $\frac{1}{2}$ respectively.

For working strength of wrought iron and steel, see pages 324 and 331.

TABLE IX.—SAFE STRENGTH OF FLAT ROLLED BARS (*concluded.*)

(Computed at 10,000 lbs. per square inch.) *

Thickness in inches.	Width in inches.									
	3½″	3¾″	4″	4¼″	4½″	4¾″	5″	5½″	6″	6½″
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
1/16	2,190	2,340	2,500	2,660	2,810	2,970	3,130	3,440	3,750	4,060
⅛	4,380	4,690	5,000	5,310	5,630	5,940	6,250	6,880	7,500	8,130
3/16	6,560	7,030	7,500	7,970	8,440	8,910	9,380	10,300	11,300	12,200
¼	8,750	9,380	10,000	10,600	11,300	11,900	12,500	13,800	15,000	16,300
5/16	10,900	11,700	12,500	13,300	14,100	14,800	15,600	17,200	18,800	20,300
⅜	13,100	14,100	15,000	15,900	16,900	17,800	18,800	20,600	22,500	24,400
7/16	15,300	16,400	17,500	18,600	19,700	20,800	21,900	24,100	26,300	28,400
½	17,500	18,800	20,000	21,300	22,500	23,800	25,000	27,500	30,000	32,500
9/16	19,700	21,100	22,500	23,900	25,300	26,700	28,100	30,900	33,800	36,600
⅝	21,900	23,400	25,000	26,600	28,100	29,700	31,300	34,400	37,500	40,600
11/16	24,100	25,800	27,500	29,200	30,900	32,700	34,400	37,800	41,300	44,700
¾	26,300	28,100	30,000	31,900	33,800	35,600	37,500	41,300	45,000	48,800
13/16	28,400	30,500	32,500	34,500	36,600	38,600	40,600	44,700	48,800	52,800
⅞	30,600	32,800	35,000	37,200	39,400	41,600	43,800	48,100	52,500	56,900
15/16	32,800	35,200	37,500	39,800	42,200	44,500	46,900	51,600	56,300	60,900
1	35,000	37,500	40,000	42,500	45,000	47,500	50,000	55,000	60,000	65,000
1 1/16	37,200	39,800	42,500	45,200	47,800	50,500	53,100	58,400	63,800	69,100
1⅛	39,400	42,200	45,000	47,800	50,600	53,400	56,300	61,900	67,500	73,100
1 3/16	41,600	44,500	47,500	50,500	53,400	56,400	59,400	65,300	71,300	77,200
1¼	43,800	46,900	50,000	53,100	56,300	59,400	62,500	68,800	75,000	81,300
1⅜	48,100	51,600	55,000	58,400	61,900	65,300	68,800	75,600	82,500	89,400
1½	52,500	56,300	60,000	63,800	67,500	71,300	75,000	82,500	90,000	97,500
1⅝	56,900	60,900	65,000	69,100	73,100	77,200	81,300	89,400	97,500	105,600
1¾	61,300	65,600	70,000	74,400	78,800	83,100	87,500	96,300	105,000	113,800
1⅞	65,600	70,300	75,000	79,700	84,400	89,100	93,800	103,100	112,500	121,900
2	70,000	75,000	80,000	85,000	90,000	95,000	100,000	110,000	120,000	130,000

* See foot-note, preceding page.

TABLE X.—SAFE TENSILE STRENGTH, IN TONS, OF COMMON SIZES OF STEEL ANGLES WITH ONE $\frac{7}{8}$-INCH HOLE FOR $\frac{3}{4}$-INCH RIVET DEDUCTED.

(Based on a working stress of 15,000 lbs. per square inch.)

Size of Angle.	Tons.	Size of Angle.	Tons.	Size of Angle.	Tons.
$6\times4\times\frac{3}{4}$	47.10	$3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{4}$	30.21	$3\times2\frac{1}{2}\times\frac{1}{2}$	15.45
$\frac{5}{8}$	39.82	$\frac{5}{8}$	25.72	$\frac{3}{8}$	11.92
$\frac{1}{2}$	32.22	$\frac{1}{2}$	21.07	$\frac{5}{16}$	10.12
		$\frac{3}{8}$	16.12	$\frac{1}{4}$	8.17
$5\times3\frac{1}{2}\times\frac{3}{4}$	38.62				
$\frac{5}{8}$	32.77	$3\frac{1}{2}\times3\times\frac{5}{8}$	23.40	$3\times2\times\frac{7}{16}$	12.15
$\frac{1}{2}$	26.70	$\frac{1}{2}$	19.20	$\frac{3}{8}$	10.50
		$\frac{3}{8}$	14.77	$\frac{5}{16}$	9.00
$5\times3\times\frac{3}{4}$	35.85			$\frac{1}{4}$	7.27
$\frac{5}{8}$	30.45	$3\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{8}$	21.07		
$\frac{1}{2}$	24.82	$\frac{9}{16}$	19.27	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{7}{16}$	12.15
$\frac{3}{8}$	18.97	$\frac{1}{2}$	17.22	$\frac{3}{8}$	10.50
		$\frac{3}{8}$	13.35	$\frac{5}{16}$	9.00
$4\times4\times\frac{3}{4}$	35.85	$\frac{1}{4}$	9.15	$\frac{1}{4}$	7.27
$\frac{3}{8}$	21.22				
		$3\times3\times\frac{5}{8}$	21.07	$2\frac{1}{2}\times2\times\frac{7}{16}$	10.50
$4\times3\frac{1}{2}\times\frac{5}{8}$	28.12	$\frac{1}{2}$	17.32	$\frac{3}{8}$	9.15
$\frac{3}{8}$	17.55	$\frac{3}{8}$	13.35	$\frac{5}{16}$	7.80
		$\frac{1}{4}$	9.15	$\frac{1}{4}$	6.30
$4\times3\times\frac{5}{8}$	25.72				
$\frac{1}{2}$	21.07				
$\frac{3}{8}$	16.12				

TABLE XI.

SECTIONAL AREA TO BE DEDUCTED FROM PLATES AND ANGLES FOR ONE ROUND HOLE.

NOTE.—Bolt-holes should be $\frac{1}{16}$ inch larger than the diameter of the bolt; rivet-holes are usually $\frac{1}{8}$ inch larger than the diameter of the rivet.*

Thickness of plate.	Diameter of Hole.																
	$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$	$\frac{15}{16}''$	$1''$	$1\frac{1}{16}''$	$1\frac{3}{16}''$	$1\frac{7}{16}''$	$1\frac{9}{16}''$
$\frac{3}{16}$	.05	.06	.07	.08	.09	.11	.12	.13	.14	.15	.16	.18	.19	.20	.23	.27	.30
$\frac{1}{4}$	.06	.08	.09	.11	.13	.14	.16	.17	.19	.20	.22	.23	.25	.27	.30	.36	.39
$\frac{5}{16}$	.08	.10	.12	.14	.16	.18	.20	.21	.23	.25	.27	.29	.31	.33	.37	.45	.49
$\frac{3}{8}$	.09	.12	.14	.16	.19	.21	.23	.26	.28	.30	.33	.35	.38	.40	.45	.54	.59
$\frac{7}{16}$	.11	.14	.16	.19	.22	.25	.27	.30	.33	.36	.38	.41	.44	.46	.52	.63	.69
$\frac{1}{2}$	.13	.16	.19	.22	.25	.28	.31	.34	.38	.41	.44	.47	.50	.53	.59	.72	.78
$\frac{9}{16}$	.14	.18	.21	.25	.28	.32	.35	.39	.42	.46	.49	.53	.56	.60	.67	.81	.88
$\frac{5}{8}$	.16	.20	.23	.27	.31	.35	.39	.43	.47	.51	.55	.59	.63	.66	.74	.90	.98
$\frac{11}{16}$	.17	.21	.26	.30	.34	.39	.43	.47	.52	.56	.60	.64	.69	.73	.82	.99	1.08
$\frac{3}{4}$	.19	.23	.28	.33	.38	.42	.47	.52	.56	.61	.66	.70	.75	.80	.89	1.08	1.17
$\frac{13}{16}$	.20	.25	.30	.36	.41	.46	.51	.56	.61	.66	.71	.76	.81	.86	.97	1.17	1.27
$\frac{7}{8}$	.22	.27	.33	.38	.44	.49	.55	.60	.66	.71	.77	.82	.88	.93	1.04	1.26	1.37
$\frac{15}{16}$	.23	.29	.35	.41	.47	.53	.59	.64	.70	.76	.82	.88	.94	1.00	1.11	1.35	1.47
1	.25	.31	.38	.44	.50	.56	.63	.69	.75	.81	.88	.94	1.00	1.06	1.19	1.44	1.56

* See also Table I., Chapter XX.

IRON WIRE.

TABLE XII.—SHOWING SIZE, WEIGHT, AND STRENGTH OF CHARCOAL-IRON WIRE.

(Trenton Iron Co.'s List.)

No. by wire gauge.	Diameter in decimals of 1 inch.	Feet to the pound.	Weight of 1 mile, in lbs.	Area of section, in decimals of 1 square inch.	Actual breaking weight of bright market wire, in lbs.	Tensile strength of bright market wire per sq. in. of section, in lbs.
00000	.450	1.863	2833.248	.15904	12,598	79,217
0000	.400	2.358	2238.878	.12566	9,955	79,220
000	.360	2.911	1813.574	.10179	8,124	79,811
00	.330	3.465	1523.861	.08553	6,880	80,437
0	.305	4.057	1301.678	.07306	5,926	81,110
1	.285	4.645	1136.678	.06379	5,226	81,925
2	.265	5.374	982.555	.05515	4,570	82,873
3	.245	6.286	839.942	.04714	3,948	83,756
4	.225	7.454	708.365	.03976	3,374	84,862
5	.205	8.976	588.139	.03301	2,839	86,000
6	.190	10.453	505.084	.02835	2,476	87,349
7	.175	12.322	428.472	.02405	2,136	88,802
8	.160	14.736	358.3008	.02011	1,813	90,153
9	.145	17.950	294.1488	.01651	1,507	91,276
10	.130	22.333	236.4384	.01327	1,233	92,916
11	.1175	27.340	193.1424	.01084	1,010	93,170
12	.105	34.219	154.2816	.00866	810	93,530
13	.0925	44.092	119.7504	.00672	631	93,900
14	.080	58.916	89.6016	.00503	474	94,234
15	.070	76.984	68.5872	.00385	372	96,701
16	.061	101.488	52.008	.00292	292	100,000
17	.0525	137.174	38.4912	.00216	222	102,777
18	.045	186.335	28.3378	.00159	169	106,289
19	.040	235.084	22.3872	.0012566	137	109,024
20	.035	308.079	17.1389	.0009621	107	111,215

The gauge given is that adopted by the Trenton Iron Company. The strengths given in the last column of the above table are based upon tests made with bright (not annealed) charcoal-iron wire. The strength of Swedish iron is about 10 per cent. less, and that of mild bessemer and ordinary crucible cast steel about 10 and 25 per cent., respectively, greater, than that of charcoal iron. Special grades of crucible cast steel vary between 30 and 100 per cent. over charcoal iron.

Annealing renders wire more pliable but less elastic, and reduces its strength about 20 or 25 per cent. Galvanizing reduces the tensile strength about 10 per cent., while tinning and coppering exert no apparent influence upon the metal. Unannealed or hard *brass wire* has about three-fourths the strength of the above table, and about one-ninth more weight.

Hard copper wire may be taken at two-thirds of the tabular strengths, and full one-seventh more in weight.

WIRE ROPES.

Two kinds of wire rope are manufactured. The most pliable variety is made of six strands of nineteen wires each, laid around a hemp heart, and is generally used for hoisting and running rope. It will wind on moderate-sized drums and pass over small sheaves.

For standing rope, guys and rigging, ropes made of six strands of twelve or seven wires each are better adapted, as they are much stiffer than rope with 19 wires to the strand. From $\frac{3}{4}$-inch diameter down to the smaller sizes this rope gives excellent service for transmitting power.

Steel ropes are in many places superseding iron ropes.

In substituting steel rope for iron rope, however, the object in view should be to gain an increased wear for the rope, rather than to reduce the size.

To be serviceable, a steel rope should be of the best obtainable quality, as ropes made from low grades of steel are inferior to good iron ropes. The constant bending and vibration to which they are subjected soon causes the poor steel to become brittle and unsafe.

Ropes are made up to three inches in diameter, both of iron and steel, upon special application.

For safe working load, allow one-fifth to one-seventh of the ultimate strength, according to speed, so as to get good wear from the rope. When substituting wire rope for hemp rope, it is good economy to allow for the former the same weight per foot which experience has approved for the latter.

Wire rope is as pliable as new hemp rope of the same strength; the former will therefore run over the same sized sheaves and pulleys as the latter. But the greater the diameter of the sheaves, pulleys or drums, the longer wire rope will last. In the construction of machinery for wire rope it will be found good economy to make the drums and sheaves as large as possible. The minimum size of drum is given in a column in Table XIII.

Experience has demonstrated that the wear increases with the speed. It is, therefore, better to increase the load than the speed.

Wire rope is manufactured either with a wire or a hemp centre, and the kind of centre wanted should be specified when placing

an order. The latter is more pliable than the former, and will wear better where there is short bending.

Wire rope must not be coiled or uncoiled like hemp rope. When mounted on a reel, the latter should be mounted on a spindle or flat turn-table to pay off the rope. When forwarded in a small coil, without reel, it should be rolled over the ground like a wheel, and the rope run off in that way. All untwisting or kinking must be avoided.

To preserve wire rope, apply raw linseed oil with a piece of sheep-skin, wool inside, or mix the oil with equal parts of Spanish brown or lamp-black.

To preserve wire rope under water or under ground, take mineral or vegetable tar, and add one bushel of fresh-slacked lime to one barrel of tar, which will neutralize the acid. Boil it well, and saturate the rope with the hot tar. To give the mixture body, add some sawdust.

In no case should *galvanized rope* be used for running rope. One day's use scrapes off the coating of zinc, and rusting proceeds with twice the rapidity.

The grooves of cast-iron pulleys and sheaves should be filled with well-seasoned blocks of hard wood, set on end, to be renewed when worn out. This end wood will save wear and increase adhesion. The smaller pulleys or rollers which support the ropes on inclined planes should be constructed on the same plan. When large sheaves run with very great velocity, the grooves should be lined with leather, set on end, or with India rubber. This is done in the case of all sheaves used in the *transmission of power* between distant points by means of rope, which frequently run at the rate of 4,000 feet per minute.

The wire ropes described above are sold by the foot.

Ropes, Hawsers, and Cables.

(HASWELL.)

Ropes of hemp fibres are laid with three or four strands of twisted fibres and run up to a circumference of twelve inches.

Hawsers are laid with three strands of rope, or with four rope strands.

Cables are laid with three strands of rope only.

Tarred ropes, hawsers, etc., have twenty-five per cent. less strength than white ropes; this is in consequence of the injury the fibres receive from the high temperature of the tar—290°.

GALVANIZED STEEL WIRE STRAND, COMPOSED OF 7 WIRES, TWISTED TOGETHER INTO A SINGLE STRAND.

FOR SMOKESTACK GUYS, SIGNAL STRAND, TROLLEY LINE SPAN WIRE AND SIMILAR PURPOSES.

Diameter.	Weight per 100 feet.	Estimated breaking strength.	Price per 100 feet.
Inch.	Pounds.	Pounds.	
½	51	8,320	$2.25
15/32	48	7,500	2.05
7/16	37	6,000	1.65
⅜	30	4,700	1.40
5/16	21	3,300	1.05
9/32	18	2,600	.90
17/64	15	2,250	.75
¼	11½	1,750	.60
7/32	8¾	1,300	.50
3/16	6½	1,000	.45
5/32	4½	700	.35
9/64	3½	525	.28
⅛	2¼	375	.22
3/32	2	320	.20

Tarred hemp and manila ropes are of about equal strength. Manila ropes have from twenty-five to thirty per cent. less strength than white ropes. Hawsers and cables, from having a less proportionate number of fibres, and from the increased irregularity of the resistance of the fibres, have less strength than ropes; the difference, varying from thirty-five to forty-five per cent., being greatest with the least circumference.

Ropes of four strands, up to eight inches, are fully sixteen per cent. stronger than those having but three strands.

Hawsers and cables of three strands, up to twelve inches, are fully ten per cent. stronger than those having four strands.

The absorption of tar in weight by the several ropes is as follows:

Bolt-rope. . . . . . . 18 per cent.	Cables . . . . 21 per cent.
Shrouding. .15 to 18 per cent.	Spun-yarn. .25 to 30 per cent.

White ropes are more durable than tarred.

The greater the degree of twisting given to the fibres of a rope, etc., the less its strength, as the exterior alone resists the greater portion of the strain.

TABLE XIII.—STRENGTH OF IRON- AND STEEL-WIRE ROPES.

MANUFACTURED BY THE JOHN A. ROEBLING'S SONS CO., NEW YORK.

Trade No.	Diam. in inches.	Weight per foot in lbs. of rope with hemp centre.	Iron.		Cast Steel.		Min. size of drum or sheave in feet.	
			Breaking strain in tons.	Proper working load in tons.	Breaking strain in tons.	Proper working load in tons.	Iron.	Cast steel.
HOISTING ROPE. WITH NINETEEN WIRES TO THE STRAND.								
1	2¼	8.00	74.00	15.00	155	31	13	8.50
2	2	6.30	65.00	13.00	125	25	12	8
3	1¾	5.25	54.00	11.00	106	21	10	7.25
4	1⅝	4.10	44.00	9.00	86	17	8	6.25
5	1½	3.65	39.00	8.00	77	15	7	5.75
5½	1⅜	3.00	33.00	6.50	63	12	7	5.50
6	1¼	2.50	27.00	5.50	52	10	6	5
7	1⅛	2.00	20.00	4.00	42	8	6	4.50
8	1	1.58	16.00	3.00	33	6	5.25	4
9	⅞	1.20	11.50	2.50	25	5	4	3.50
10	¾	0.88	8.64	1.75	18	3.5	4	3
10¼	⅝	0.60	5.13	1.25	12	2.5	3	2.25
10½	9/16	0.48	4.27	0.75	9	1.5	2.75	1.75
10¾	½	0.39	3.48	0.50	7	1	2.25	1.50
10*a*	7/16	0.29	3.00	0.37	5.5	0.75	2	1.25
10⅞	⅜	0.23	2.50	0.25	4.5	0.5	1.50	1
STANDING ROPE FOR GUYS AND RIGGING. WITH SEVEN WIRES TO THE STRAND.								
11	1½	3.37	36.00	9.00	62	13	13	8.50
12	1⅜	2.77	30.00	7.50	52	10	12	8
13	1¼	2.28	25.00	6.25	44	9	10.75	7.25
14	1⅛	1.82	20.00	5.00	36	7.50	9.50	6.25
15	1	1.50	16.00	4.00	30	6	8.50	5.75
16	⅞	1.12	12.30	3.00	22	4.50	7.50	5
17	¾	0.92	8.80	2.25	17	3.50	6.75	4.50
18	11/16	0.70	7.60	2.00	14	3	6	4
19	⅝	0.57	5.80	1.50	11	2.25	5.25	3.50
20	9/16	0.41	4.10	1.00	8	1.75	4.50	3
21	½	0.31	2.83	0.75	6	1.50	4	2.50
22	7/16	0.23	2.13	0.50	4.50	1.25	3.50	2.25
23	⅜	0.21	1.65	———	4	1	2.75	2
24	5/16	0.16	1.38	———	3	0.75	2.50	1.75
25	9/32	0.125	1.03	———	2	0.50	2.25	1.50

NOTE.—A valuable pamphlet on wire rope for the transmission of power may be obtained from the Trenton Iron Co., of Trenton, N. J.

To compute the Strain that can be borne with Safety by New Ropes, Hawsers, and Cables, Deduced from the Experiments of the Russian Government upon the Relative Strength of Different Circumferences of Ropes, Hawsers, etc.

The United States Navy test is 4,200 *pounds for a white rope, of three strands of best Riga hemp, of one and three-fourths inches in circumference (i.e.,* 17,000 *pounds per square inch); but in the following table* 14,000 *pounds is taken as the unit of strain that can be borne with safety.*

RULE.—Square the circumference of the rope, hawser, etc., and multiply it by the following units for ordinary ropes, etc.

TABLE XIV.—SHOWING THE UNITS FOR COMPUTING THE SAFE STRAIN THAT MAY BE BORNE BY ROPES, HAWSERS, AND CABLES.

Description.	Ropes.				Hawsers.		Cables.	
	White.		Tarred.		White, 3 str'ds.	Tar'd, 3 strands.	White, 3 str'ds.	Tar'd, 3 strands.
	3 strands.	4 strands.	3 strands.	4 strands.				
Circumference in inches. . .	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
White rope, 2.5 to 6 ins. . .	1,140	1,330	—	—	600	—	—	—
White rope, 6 to 8 ins	1,090	1,260	—	—	570	—	510	—
White rope, 8 to 12 ins. . . .	1,045	880	—	—	530	—	530	—
White rope, 12 to 18 ins. . .	—	—	—	—	550	—	550	—
White rope, 18 to 26 ins. . .	—	—	—	—	—	—	560	—
Tarred rope, 2.5 to 5 ins. . .	—	—	855	1,005	—	460	—	—
Tarred rope, 5 to 8 ins. . . .	—	—	825	940	—	480	—	—
Tarred rope, 8 to 12 ins. . .	—	—	780	820	—	505	—	505
Tarred rope, 12 to 18 ins .	—	—	—	—	—	—	—	525
Tarred rope, 18 to 26 ins. .	—	—	—	—	—	—	—	550
Manila rope, 2.5 to 6 ins. ..	810	950	—	—	440	—	—	—
Manila rope, 6 to 12 ins. . .	760	835	—	—	465	—	510	—
Manila rope, 12 to 18 ins. .	—	—	—	—	—	—	535	—
Manila rope, 18 to 26 ins. .	—	—	—	—	—	—	560	—

When it is required to ascertain the weight or strain that can be borne by ropes, etc., in general use, the above units should be reduced *one-third,* in order to meet the reduction of their strength by chafing and exposure to the weather.

TABLE XV.—STRENGTH AND WEIGHT OF MANILA ROPE.

Diameter.	Circumference.	Weight per foot.	Breaking load.		Diameter.	Circumference.	Weight per foot.	Breaking load.	
ins.	ins.	lbs.	lbs.	tons.	ins.	ins.	lbs.	lbs.	tons.
0.239	¾	.019	560	0.280	1.92	6	1.19	25,536	12.77
0.318	1	.033	784	0.39	2.07	6½	1.39	29,120	14.56
0.477	1½	.074	1,568	0.78	2.23	7	1.62	32,704	16.35
0.636	2	.132	2,733	1.36	2.39	7½	1.86	36,288	18.14
0.795	2½	.206	4,278	2.14	2.55	8	2.11	39,872	19.93
0.955	3	.297	6,115	3.06	2.86	9	2.67	47,040	23.52
1.11	3½	.404	8,534	4.27	3.18	10	3.30	54,208	27.10
1.27	4	.528	11,558	5.78	3.50	11	3.99	61,376	30.69
1.43	4½	.668	14,784	7.39	3.82	12	4.75	68,544	34.27
1 59	5	.825	18,368	9.18	4.14	13	5.58	75,712	37.86
1.75	5½	.998	21,952	10.97	4.45	14	6.47	82,880	41.44

Working Loads.—For manila ropes from 1 to 1¾ ins. diam., running at different speeds over sheaves of the diams. stated, Mr. C. W. Hunt (Trans. Am. Soc. Mechl. Engrs., Vol. XXIII, 1901) gives a table embodying approximately the following results of experience. Working load = C × ultimate strength of new rope, as given in above table. D = minimum diam. of sheave in ins.

Speed	ft. per min.	as for work on	C	1″ rope D	1¾″ rope D
Slow	50 to 100	derrick, crane, quarry	0.140	8	14
Medium	150 to 300	wharf, cargo	0.056	12	18
Rapid	400 to 800		0.028	40	70

Such ropes wear out rapidly. A rope 1½ ins. diam. wears out in lifting from 7,000 to 10,000 tons of coal. On the other hand, 1½ inch transmission ropes, running 5000 ft. per min. and carrying 1000 H. P. over sheaves 5 ft. and 17 ft. in diam. last for years.

TABLE XVI.—WEIGHT AND PROOF STRENGTH OF CHAIN.

MANUFACTURED BY THE NEW JERSEY STEEL AND IRON COMPANY.

Stud chain.			Short-link chain.			X. B. crane chain.
Size.	Average weight per fathom.	Proof.	Size.	Average weight per fathom.	Proof.	Proof.
inches.	lbs.	tons.	inches.	lbs.	tons.	tons.
3/4	33	10	3/16	2 3/4	1/4	—
13/16	38	12	1/4	5	1/2	—
7/8	43	14	5/16	7	1	—
15/16	50	16	3/8	9 1/2	2	3
1	58	18	7/16	12	2 1/2	4
1 1/16	65	20	1/2	15	3 1/2	4 1/2
1 1/8	72	23	9/16	19	4 1/2	5 1/2
1 3/16	80	26	5/8	25	5 1/2	7
1 1/4	88	28	11/16	30	7	8 1/2
1 5/16	98	31	3/4	35	8	10
1 3/8	110	34	13/16	40	9 1/2	11 1/2
1 7/16	114	37	7/8	47	11	13
1 1/2	127	41	15/16	54	12 1/2	14 1/2
1 9/16	138	44	1	61	14	16
1 5/8	150	48	1 1/16	69	16	19
1 11/16	157	52	1 1/8	76	18	21
1 3/4	170	56	1 3/16	85	20	23
1 13/16	184	60	1 1/4	95	22	25
1 7/8	200	64	1 5/16	103	24	27
1 15/16	214	68	1 3/8	113	26	29
2	230	72	1 7/16	123	28	31
2 1/3	250	80	1 1/2	133	30	33
2 1/4	290	88				

Strength of Old Iron.—A square link 12 inches broad, 1 inch thick, and about 12 feet long was taken from the Kieff Bridge, then 40 years old, and tested in comparison with a similar link which had been preserved in the stock-house since the bridge was built. The following is a record of a mean of four longitudinal test pieces, $1 \times 1\frac{1}{8} \times 8$ inches, taken from each link.

	Old link from bridge.	New link from storehouse.
Tensile strength per square inch, tons...	21.8	22.2
Elastic limit per square inch, tons......	11.1	11.9
Elongation, per cent..................	14.05	18.42
Contraction, per cent.................	17.35	18.75

(*The Mechanical World*, London.)

CHAPTER XII.

RESISTANCE TO SHEARING.—RIVETED JOINTS.

STRENGTH OF PINS IN IRON AND STEEL TRUSSES.—STRENGTH OF BOLTS IN WOODEN TRUSSES AND GIRDERS.

By shearing is meant the pushing of one part of a piece by the other. Thus in Fig. 1, let *abcd* be a beam resting upon the supports *SS*, which are very near together. If a sufficiently

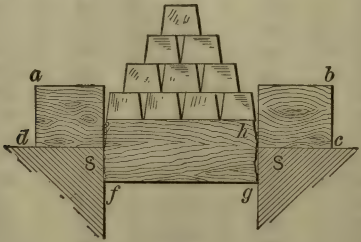


Fig. 1

heavy load were placed upon the beam, it would cause the beam to break, not by bending, but by pushing the whole central part of the beam through between the ends, as represented in the figure. This mode of fracture is called "shearing."

Shearing stresses exist whenever two forces acting like a pair of shears tend to cut a body between them.

When two bars of steel or iron are connected together by a rivet, as in Fig. 2, the stresses in the bars acting as indicated by the arrow-heads, the tendency is to shear the rivet at the joint between the two bars, as though it were cut by a pair of shears. When the shearing stresses tend to shear the piece only at one place, as in Fig. 2, the piece is said to be in *single shear;* when the stresses tend to shear the piece at two sections, as in Fig. 1, the piece is in "double shear."

The resistance of a body to shearing is, like its resistance to tension, directly proportional to the area to be sheared. Hence, if we denote the safe resistance of one square inch of the mate-

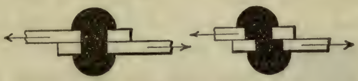


Fig. 2

rial to shearing by F, we shall have as the safe resistance to shearing,

$$\text{Safe shearing strength} = \text{area to be sheared} \times F, \qquad (1)$$

A piece of timber may be sheared either longitudinally or transversely; and, as the resistance is not the same in both cases, the value of F will be different in the two cases. Hence, in substituting values for F, we must distinguish whether the force tends to shear the piece longitudinally (lengthwise) or transversely (across).

Table I. gives the average working values of F in building construction as recommended by the best authorities, and as used by structural engineers.

TABLE I.—SAFE RESISTANCE TO SHEARING, IN LBS. PER SQUARE INCH, FOR IRON, STEEL, AND WOOD.

Materials.	Values for F.	
Cast iron	....	6,000
	Wrought iron.	Rolled steel.
Webs of rolled beams and riveted girders	6,000 to 9,000	7,000 to 10,000
Bolts and field-driven rivets	6,000	7,500 *
Rivets, shop-driven	7,500	7,500 to 10,000
Pins	7,500	7,500 to 9,000
Woods.	With grain.	Across grain
Cedar	....	400
Chestnut	125	400
Hemlock	80	600
Oak, white	150	1,000
Pine, Georgia yellow	125	1,200
Pine, Oregon	125	900
Pine, Norway	90	750
Pine, white	80	500
Redwood (Califorina)	70	500
Spruce	90	750
Whitewood	60.	....

* For bolts in the connection-angles of beams, a shearing stress of 10,000 lbs. is usually allowed, and the author believes that the same unit stress may be used for bolts in the joints of wooden trusses.

There are but few cases of architectural construction in which the resistance of wood to shearing has to be provided for. The one most frequently met with is at the end of the tie-beam in wooden trusses.

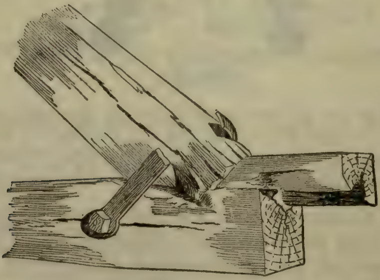


Fig. 3

Fig. 3 shows the shearing of the end of the tie-beams due to the thrust of the rafter, the drawing being made from a photograph of an actual instance.

There is always a tendency for beams to shear vertically at the points of support, as in Fig. 1, but it is very rare that wooden beams are subject to dangerous stresses from shearing. It could only happen in the case of a very short beam very heavily loaded. The vertical shearing stresses in a beam are explained in Chapter XX.

A very long beam might also possibly fail by shearing longitudinally, but such failure is not likely to occur under the safe load given by either the formula for strength or the formula for stiffness. Other common instances in which failure by shear-

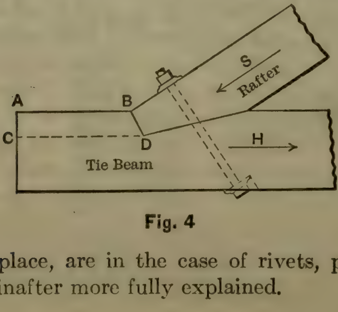


Fig. 4

ing may take place, are in the case of rivets, pins, and bolts, which are hereinafter more fully explained.

Example of Shear at End of Tie-beam.—In the case of the truss joint, Fig. 4, the rafter exerts a thrust which tends to push or shear off the piece $ABCD$, and the area of the section along CD should offer enough resistance to keep the rafter in place. This area is equal to CD times the breadth of the tie-beam; and, as the breadth is fixed, we have to determine the length, CD. If we let H denote the horizontal thrust of the rafter, or the tension in the tie-beam, by a simple deduction from formula (1), we have the rule,

$$\text{Length of } CD \text{ in inches} = \frac{H}{\text{breadth of beam} \times F}, \qquad (2)$$

F, in this case, being the resistance to shearing longitudinally.

Example I.—The horizontal thrust of a rafter is 20,000 pounds, the tie-beam is of Oregon pine, and is ten inches wide: how far should the beam extend beyond the point D?

Ans. In this case $H = 20{,}000$ pounds, and from Table I. we find that $F = 125$. Then

$$CD = \frac{20{,}000}{10 \times 125}, \text{ or 16 inches.}$$

Practically a large part of the thrust is generally taken up by an iron bolt or strap passed through or over the foot of the rafter and tie-beam, in order to keep the rafter in place. As the bolt and shoulder seldom act together, the length of the tie-beam at CD should either be made long enough to resist the entire thrust without help from the bolt, or else the bolt or strap should be strong enough to resist the entire thrust. The designing of such joints is more fully considered on pages 382 to 397.

RIVETED JOINTS.

The most common method of uniting pieces of wrought iron or steel in framed structures is by means of rivets, and that the structures shall be equally strong in all its parts it is essential that the joints shall be carefully designed.

A rivet is a piece of metal with a solid head at one end, and a long circular shank.

Riveting consists of heating the rivet, passing it through the holes in the plates to be united while hot, and then forging another solid head out of the projecting end of the shank.

The hammering causes the heated shank to fill all parts of the holes, and the contraction of the metal, as it cools, draws the heads together, thus firmly forcing and holding the pieces together.

Rivets are generally made either of mild steel or the best wrought iron, the latter being the most reliable. The rivet-heads are made in four ways, as shown in Fig. 5.

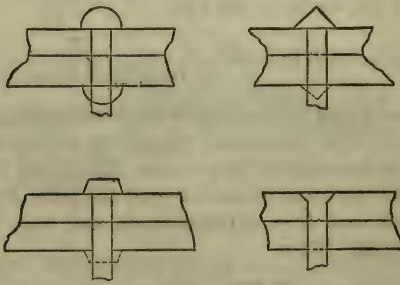


Fig. 5.

The first shape is the one generally used. The second and third are used only for their appearance; and the fourth, or counter-sunk head, is only used when a smooth surface is desirable, as over a bearing plate.

The exact sizes of heads, shapes, etc., of rivets vary in different mills.

When the size of rivet is specified the hole is always made $\frac{1}{16}$ inch larger; but the rivet is generally designated by the size of the hole.

Pitch.—The distance between the centres of the rivets, in the line of riveting, is called the *pitch*. This (for practical reasons) should never be less than $2\frac{1}{2}$ diameters; nor should the centre of the hole (if possible) be nearer to any edge than $1\frac{1}{2}$ diameters. In angle work, however, it is often necessary to make the distance from the edge less than the above, but in thick plates it should always be more. In drilled work the pitch might be reduced to 2 diameters. If rivet-heads are counter-sunk the pitch should be increased according to the amount of metal cut away, to make room for the rivet-head.

Rivet-holes are generally made by punching, by a powerful steam-punch, as this is much the cheapest method. The best

way to make the holes is to drill them after the pieces are bolted or clamped together.

Punching makes a ragged and irregular hole, and injures the metal about the hole, causing a loss of strength to the remaining portion of the metal of 15 per cent. in wrought-iron, and often 35 per cent. in steel.

Besides this, in punching there is liability of cracking the plate, and of not having the holes in the two plates that are to be united come exactly opposite each other.

The hardening of the metal by punching also decreases the ductility of the pieces.

The injury done by punching in steel plates may be almost entirely removed, however, by annealing, and in first-class work this should always be done.

In drilled work there is no loss, and the holes are not only accurately located, but accurately cut, and the strength of the remaining fibres is even increased from 12 to 25 per cent.

The cost of drilling, however, is very great, so that it is not likely to be employed, except in making the joints in trusses and connecting tie-bars, where the number of rivets is not great.

A medium course between punching and drilling is to punch the holes a size smaller than desired, and then drill or ream them to actual size, when partially secured together. The loss of strength by this method will be very slight.

In most cases, however, the architect will have to be satisfied with punched holes, and must, therefore, allow sufficient metal to make good any damage done, or for any inaccuracies.

In driving and heading the rivet, however, machine riveting is much better than hand riveting, as a greater pressure is used, and the metal more completely fills the hole.

In designing riveted work, whether to be hand or machine riveted, the architect must bear in mind the necessity of placing the rivets so that they can be inserted in the holes from one side and hammered from the other; and for machine work, that the machine can reach them. Thus, the minimum distance from the inside face of one leg of an angle iron to centre of nearest rivet-hole in other leg should be at least $1\frac{1}{8}$ inch for $\frac{7}{8}$-inch rivets, 1 inch for $\frac{3}{4}$-inch rivets, $\frac{7}{8}$ inch for $\frac{5}{8}$-inch rivets, $\frac{13}{16}$ inch for $\frac{1}{2}$-inch rivets; and, if possible, these distances should be increased.

Failure of Riveted Joints. — Riveted joints may yield in any one of five ways:

1st. By the crushing of the plate in front of the rivets (Fig. 6).

2d. By the shearing of the rivets (Fig. 7).
3d. By the tearing of the plate between the rivet-holes (Fig. 8).
4th. By the rivet breaking through the plate (Fig. 9).
5th. By the rivet shearing out the plate in front of it.

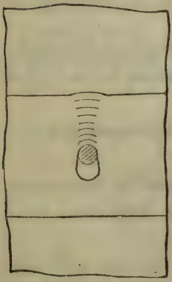


Fig. 6


Fig. 7

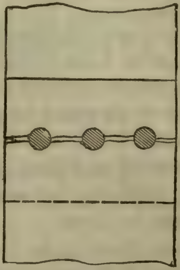


Fig. 8

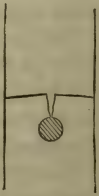


Fig. 9

The two latter cases are likely to occur only in the case of a single riveted lap-joint.

To design a riveted joint so that it will not break in either of these ways, it is, therefore, necessary to calculate for the *shearing strength* of the rivets, for the *crushing strength of the plates* joined, and to *space the rivets* far enough apart that the metal will not tear between the rivets.

The process of designing a riveted joint practically consists in first assuming the size of rivet to be used, and then calculating the number required to resist shearing and to prevent the crushing of the plates joined, and then using the larger number. They are then spaced by the rule that the pitch shall not be less than $2\frac{1}{2}$ diameters, nor more than 16 times the thickness of the thinnest plate at the joint, and the distance from the centre of the rivet to end of the plate should not be less than $1\frac{1}{2}$ diameters. The following table gives the sizes of rivets to be preferred for different thicknesses of plates:

For plates from $\frac{1}{4}$ inch to $\frac{7}{16}$ inch thick, use rivet-holes $\frac{5}{8}$ inch in diameter.

For plates from $\frac{1}{2}$ inch to $\frac{5}{8}$ inch thick, use rivet-holes $\frac{3}{4}$ inch in diameter.*

For plates from $\frac{11}{16}$ inch to $\frac{13}{16}$ inch thick use rivet-holes $\frac{7}{8}$ inch in diameter.

* In truss work $\frac{3}{4}''$ rivets are generally used for thicknesses of plates and angles from $\frac{5}{16}$-inch to $\frac{13}{16}$-inch.

For plates from $\frac{7}{8}$ inch to 1 inch thick, use rivet-holes 1 inch in diameter.

The number of rivets required to *resist shearing* can be easily determined by dividing the total amount of strain by the number opposite the size of the rivet, in the fourth column of Tables II. and III., pages 371 and 372, if the rivet is in single shear; and if in double shear, take one-half the number of rivets.

To find the number of rivets required to prevent crushing, divide the total amount of strain by the bearing value of the rivet given in these tables.

Note.—Table III. should only be used for the connections of steel floor beams and roof trusses where the usual loads are to be supported; for riveted girders and live loads, or where only actual loads have been provided for, Table II. should be used. The heavy zigzag line in the tables indicates the limit at which the bearing value exceeds single shear. All values above these lines are in excess of single shear; all values below are less than single shear.

The principal cases in which riveted joints occur in building construction are: 1. In the joints of wrought-iron trusses. 2. Splicing of tie-bars. 3. In the connecting angles of floor beams. 4. In riveted girders.

Splicing of Tie-bars.

Tie-bars may be spliced in three ways.

1st. By a lap-joint, as shown in Fig. 10.

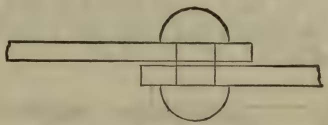


Fig. 10

2d. By a single cover plate, as shown in Fig. 11.

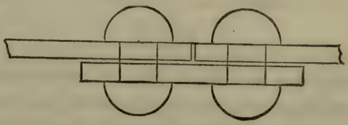


Fig. 11

3d. By two cover plates, as in Fig. 12.

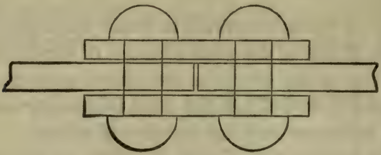


Fig. 12

In Figs. 10 and 11 the rivets are in single shear; in Fig. 12 they are in double shear. The last method is much the best. although it is also the most expensive. The cover plates should always be the full width of the bars connected, and $\frac{1}{16}$ inch more in thickness for the two plates, or for one single plate.

For lapped joints, which is the most common joint used, the rivets should be arranged as in Fig. 13, in which case the plates

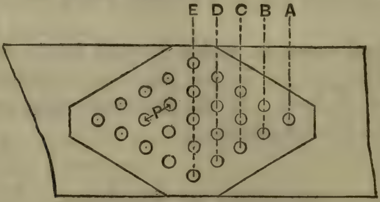


Fig. 13

are only weakened by the width of one rivet-hole, at A. At B, two rivet-holes are lost, but the strain has been reduced by an amount equal to the value of one rivet-hole and so on.

If the plates are narrow and thick, the rivets may be arranged as in Figs. 14 or 15.

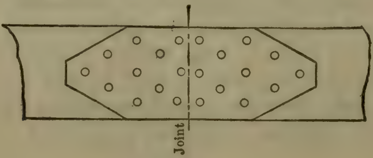


Fig. 14

Where cover plates are used, Fig. 15 is the best arrangement, for by it the cover plates are weakened by only two rivet-holes (the ones nearest the joint); while in Fig. 14 the cover plates

are weakened by three holes nearest the joint, and, consequently, must be made thicker.

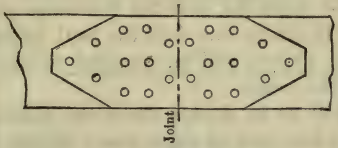


Fig. 15

When rivets are arranged in rows, it is called chain riveting; when rivets are arranged to come opposite the space between the preceding rivets, they are said to be staggered, as in Figs. 13 and 14.

In designing riveted joints care must be exercised not to weaken the plates any more than is absolutely necessary.

EXAMPLE 2.—*A* 12″ × ½″ *tie-bar is so long that it has to be made in two pieces with a splice; the strain on the piece is* 65,000 *lbs. How many rivets will be required?*

Ans. We will assume that the joint is to be a lapped joint, as in Fig. 13, and that we will use ¾-inch rivets.

From Table II. we find that the resistance of a ¾-inch rivet to single shear is 3,310 lbs. and the bearing value for a half-inch plate 5,630 lbs. Dividing the strain, 65,000 lbs., by the smaller of these two quantities, 3,310, we find we shall require 20 rivets; but as 20 rivets will not give us the arrangement we wish, we will use 25, as in Fig. 13. The distance, P, between the centres of rivets measured on the slant should be at least 2½ diameters, or 2½ × ¾ inch = 1⅞ inches, or, we will say, 2 inches.

Beam Connections.

EXAMPLE 3.—*A* 10-*inch*, 30-*lb. standard steel beam having a span of* 3½ *feet supports a load at the centre of* 40,000 *lbs. and is framed at one end to a* 15-*inch* 50-*lb. steel beam; how many* ¾-*inch rivets will be required in the connection?*

Ans. From the table giving the properties of standard steel I-beams, we find the web thickness of the 10-inch beam to be 0.455(7/16) inch, and of the 15-inch beam to be 0.558 inch. The standard connection angle for a 10-inch beam (see Chap. XV.) is 6 × 4 × ⅜″. The number of rivets required in the 10-inch beam

will therefore be determined either by the bearing resistance of the web of the 10-inch beam, or by the resistance to shearing. From Table III. we find the bearing value of a $\frac{3}{4}$-inch rivet on a $\frac{7}{16}$-inch plate to be 5,890 lbs. and the resistance to single shear to be 4,420 lbs. Dividing one half the load, or 20,000 lbs., by 5,890, we find that 4 rivets will be required to sustain the load without crushing the web.

As the rivets will be in double shear the resistance of each rivet will be 8,836 lbs., and of the 4 rivets 35,344 lbs., which is in excess of the load; hence 4 rivets will be required in the 10-inch beam. In the 15-inch beam the number of rivets will be determined by the shearing value, as here the rivets are in single shear. 20,000 lbs. divided by 4,418 requires 5 rivets, or say 3 in each angle. To accommodate the 4 rivets in the 10-inch beam, the connection angle should be $6\frac{1}{2}$ inches long.

The standard connection for 10-inch beams shows 3 rivets in each flange, but the load which we have assumed is greatly in excess of that for which the standard connection is designed. The maximum safe distributed load for this beam for a 10-ft. span is 28,630 lbs., or 14,315 lbs. at each end, for which 3 rivets are ample.

EXAMPLE 4.—*One end of a* 10 × 12 *wooden beam is supported on a* 4 × 4 × $\frac{1}{2}$-*inch angle bracket, riveted to the web of an* 18-*inch* 60-*lb. steel beam; the load on the wooden beam is* 18,000 *lbs.; how many* $\frac{3}{4}$-*inch rivets will be required in the bracket?*

Ans. As the rivets are in single shear, and the web and angle are each $\frac{1}{2}$-inch thick, the number of rivets will be determined by the resistance to shearing, that being less than the bearing value. The load to be supported by the bracket will be one-half the load on the beam, or 9,000 lbs. Dividing this by 4,420, we find that two $\frac{3}{4}$-inch rivets are not quite sufficient, and we must therefore use either three $\frac{3}{4}$-inch rivets, or two $\frac{7}{8}$-inch rivets. The $\frac{3}{4}$-inch rivets should be placed 4 inches on centres, and the $\frac{7}{8}$-inch rivets 6 inches.

Rivets in Plate Girders.—The methods for proportioning the rivets to resist the various strains in plate girders are explained in detail in Chapter XX.

Bending Moment in Rivets.—While pins should always be computed for resistance to cross breaking, it is not the custom to consider the bending moment in rivets; as in a well-riveted joint it is practically impossible to produce any bending of the rivet, neither do the tests on riveted joints show any signs of the

SHEARING AND BEARING VALUES OF RIVETS. II.

FOR RIVETED GIRDERS AND WROUGHT IRON.

Diameter of rivet, in inches.		Area of rivet.	Single shear at 7,500 lbs. per sq. inch.	Bearing value for different thicknesses of plate at 15,000 lbs. per square inch. (=Diameter of rivet × thickness of plate × 15,000 lbs.)										
Fraction.	Decimal.			1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
3/8	.375	.1104	828	1,410										
7/16	.4375	.1503	1,130	1,640	2,050									
1/2	.5	.1963	1,470	1,880	2,340	2,810								
9/16	.5625	.2485	1,860	2,110	2,640	3,160	3,690							
5/8	.625	.3068	2,300	2,340	2,930	3,520	4,100							
11/16	.6875	.3712	2,780	2,580	3,220	3,870	4,510	5,160						
3/4	.75	.4418	3,310	2,810	3,520	4,220	4,920	5,630	6,330					
13/16	.8125	.5185	3,890	3,050	3,810	4,570	5,330	6,090	6,860	7,620				
7/8	.875	.6013	4,510	3,280	4,100	4,920	5,740	6,560	7,380	8,200				
15/16	.9375	.6903	5,180	3,520	4,390	5,270	6,150	7,030	7,910	8,790	9,670			
1	1.0	.7854	5,890	3,750	4,690	5,620	6,560	7,500	8,440	9,380	10,310	11,250		
1 1/16	1.0625	.8866	6,650	3,980	4,980	5,980	6,970	7,970	8,960	9,960	10,960	11,950	12,950	
1 1/8	1.125	.9940	7,460	4,220	5,270	6,330	7,380	8,440	9,490	10,550	11,600	12,660	13,710	14,770
1 3/16	1.1875	1.1075	8,310	4,450	5,570	6,680	7,790	8,910	10,020	11,130	12,250	13,360	14,470	15,590

SHEARING AND BEARING VALUES OF RIVETS. III.

FOR STEEL BEAM CONNECTIONS, AND JOINTS IN STEEL ROOF TRUSSES.

Diameter of rivet in inches.		Area of rivet.	Single shear, 10,000 lbs. per sq. inch.	Bearing value for different thicknesses of plate at 18,000 lbs. per square inch. (= Diameter of rivet × thickness of plate × 18,000 lbs.)										
Fraction.	Decimal.			$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$
$\frac{3}{8}$	.375	.1104	1,100	1,680										
$\frac{7}{16}$	.4375	.1503	1,500	1,960	2,450									
$\frac{1}{2}$	.5	.1963	1,960	2,240	2,800	3,370								
$\frac{9}{16}$	.5625	.2485	2,480	2,520	3,150	3,790	4,420							
$\frac{5}{8}$	.625	.3068	3,060	2,800	3,500	4,210	4,910							
$\frac{11}{16}$	.6875	.3712	3,710	3,090	3,860	4,640	5,400	6,180						
$\frac{3}{4}$	.75	.4418	4,420	3,370	4,210	5,060	5,890	6,740	7,580					
$\frac{13}{16}$	.8125	.5185	5,180	3,650	4,560	5,480	6,390	7,300	8,210	9,120				
$\frac{7}{8}$	.875	.6013	6,010	3,930	4,910	5,900	6,880	7,860	8,840	9,820				
$\frac{15}{16}$	.9375	.6903	6,900	4,210	5,260	6,330	7,370	8,420	9,470	10,520	11,590			
1	1.000	.7854	7,850	4,500	5,620	6,750	7,860	9,000	10,120	11,240	12,370	13,500		
$1\frac{1}{16}$	1.0625	.8866	8,860	4,780	5,970	7,170	8,360	9,560	10,750	11,940	13,140	14,340	15,530	
$1\frac{1}{8}$	1.125	.9940	9,940	5,060	6,320	7,590	8,850	10,120	11,380	12,640	13,910	15,180	16,440	17,700
$1\frac{3}{16}$	1.1875	1.1075	11,070	5,340	6,670	8,010	9,350	10,680	12,010	13,340	14,680	16,020	17,360	18,700

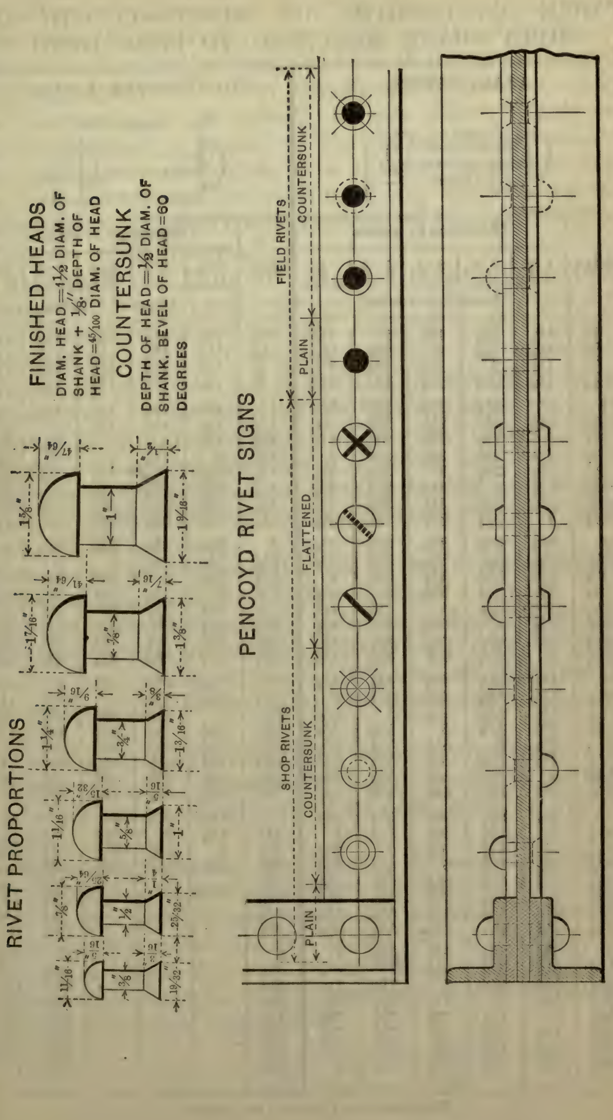
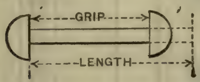

RIVET PROPORTIONS
FINISHED HEADS
DIAM. HEAD = 1½ DIAM. OF SHANK + ⅛". DEPTH OF HEAD = 45/100 DIAM. OF HEAD
COUNTERSUNK
DEPTH OF HEAD = ½ DIAM. OF SHANK. BEVEL OF HEAD = 60 DEGREES
11/16"
3/8"
19/32"
7/8"
1/2"
25/32"
1 1/16"
5/8"
1"
1 1/4"
3/4"
1 3/16"
1 7/16"
7/8"
1 3/8"
1 5/8"
1"
1 9/16"
PENCOYD RIVET SIGNS
SHOP RIVETS
FIELD RIVETS
PLAIN
COUNTERSUNK
FLATTENED
PLAIN
COUNTERSUNK

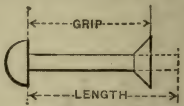
TABLE IV.—LENGTHS OF RIVETS—LENGTH OF RIVET SHANK REQUIRED TO FORM HEAD.

PLAIN RIVETS.



Grip.	Diameter in inches. 1/2	5/8	3/4	7/8	1
	Length in inches.				
½	1½	1¾	1⅞	2	2⅛
⅝	1⅝	1⅞	2	2⅛	2¼
¾	1¾	2	2⅛	2¼	2⅜
⅞	1⅞	2⅛	2¼	2⅜	2½
1	2	2¼	2⅜	2½	2⅝
1⅛	2⅛	2⅜	2½	2⅝	2¾
1¼	2¼	2½	2⅝	2¾	2⅞
1⅜	2⅜	2⅝	2¾	2⅞	3
1½	2⅝	2⅞	3	3⅛	3¼
1⅝	2¾	3	3⅛	3¼	3⅜
1¾	2⅞	3⅛	3¼	3⅜	3½
1⅞	3	3¼	3⅜	3½	3⅝
2	3⅛	3⅜	3½	3⅝	3¾
2⅛	3¼	3½	3⅝	3¾	3⅞
2¼	3⅜	3⅝	3¾	3⅞	4
2⅜	3½	3¾	3⅞	4	4⅛
2½	3⅝	3⅞	4	4⅛	4¼
2⅝	3¾	4	4⅛	4¼	4⅜
2¾	3⅞	4⅛	4¼	4⅜	4½
2⅞	4	4¼	4⅜	4½	4⅝
3	4¼	4½	4⅝	4¾	4⅞
3⅛	4⅜	4⅝	4⅞	4⅞	5
3¼	4½	4¾	4⅞	5	5⅛
3⅜	4⅝	4⅞	5	5⅛	5¼
3½	4¾	5	5⅛	5¼	5⅜
3⅝	4⅞	5⅛	5¼	5⅜	5½
3¾	5	5¼	5⅜	5½	5⅝
3⅞	5⅛	5⅜	5½	5⅝	5¾
4	5¼	5½	5⅝	5¾	5⅞
4⅛	5⅜	5⅝	5¾	5⅞	6
4¼	5½	5¾	5⅞	6	6⅛
4⅜	5⅝	5⅞	6	6⅛	6¼
4½	5⅞	6⅛	6¼	6⅜	6½
4⅝	6	6¼	6⅜	6½	6⅝
4¾	6⅛	6⅜	6½	6⅝	6¾
4⅞	6¼	6½	6⅝	6¾	6⅞
5	6⅜	6⅝	6¾	6⅞	7
5⅛	6½	6¾	6⅞	7	7⅛
5¼	6⅝	6⅞	7	7⅛	7¼
5⅜	6¾	7	7⅛	7¼	7⅜
5½	7	7¼	7⅜	7½	7⅝
5⅝	7⅛	7⅜	7½	7⅝	7¾
5¾	7¼	7½	7⅝	7¾	7⅞
5⅞	7⅜	7⅝	7¾	7⅞	8

COUNTERSUNK RIVETS.



Grip.	Diameter in inches. 1/2	5/8	3/4	7/8	1
	Length in inches.				
½	1⅛	1¼	1¼	1⅜	1⅜
⅝	1¼	1⅜	1⅜	1½	1½
¾	1⅜	1½	1½	1⅝	1⅝
⅞	1½	1⅝	1⅝	1¾	1¾
1	1⅝	1¾	1¾	1⅞	1⅞
1⅛	1¾	1⅞	1⅞	2	2
1¼	1⅞	2	2	2⅛	2⅛
1⅜	2	2⅛	2⅛	2¼	2¼
1½	2⅛	2¼	2⅜	2⅜	2½
1⅝	2¼	2⅜	2½	2½	2⅝
1¾	2⅜	2½	2⅝	2⅝	2¾
1⅞	2½	2⅝	2¾	2¾	2⅞
2	2⅝	2¾	2⅞	2⅞	3
2⅛	2¾	2⅞	3	3	3⅛
2¼	2⅞	3	3⅛	3⅛	3¼
2⅜	3	3⅛	3¼	3¼	3⅜
2½	3⅛	3¼	3⅜	3⅜	3½
2⅝	3¼	3⅜	3½	3½	3⅝
2¾	3⅜	3½	3⅝	3⅝	3¾
2⅞	3½	3⅝	3¾	3¾	3⅞
3	3¾	3¾	3⅞	4	4⅛
3⅛	3⅞	3⅞	4	4⅛	4¼
3¼	4	4⅛	4⅛	4¼	4⅜
3⅜	4⅛	4¼	4¼	4⅜	4½
3½	4¼	4⅜	4⅜	4½	4⅝
3⅝	4⅜	4½	4½	4⅝	4¾
3¾	4½	4⅝	4⅝	4¾	4⅞
3⅞	4⅝	4¾	4¾	4⅞	5
4	4¾	4⅞	5	5	5⅛
4⅛	4⅞	5	5⅛	5⅛	5¼
4¼	5	5⅛	5¼	5¼	5⅜
4⅜	5⅛	5¼	5⅜	5⅜	5½
4½				5½	5⅝
4⅝				5⅝	5¾
4¾				5¾	5⅞
4⅞				5⅞	6
5				6	6⅛
5⅛				6⅛	6¼
5¼				6¼	6⅜
5⅜				6⅜	6½
5½				6½	6⅝
5⅝				6⅝	6¾
5¾				6¾	6⅞
5⅞				6⅞	7

For weight of rivets see Index.

rivets breaking in that way. Mr. C. W. Bryan, engineer for the *Edge Moor Bridge Works,* says: "Rivets will fail by flexure only in those cases of bad designing where the rivets are long and it is impossible to drive them tight enough to have them upset and completely fill the holes." He also adds: "*Rivets are never proportioned for flexure.*" * The only person that considers the bending moment on rivets, so far as the author has been able to learn, is Mr. Louis DeCoppet Berg, who has taken up the subject of riveted joints most elaborately in Chapter IX. of Part II. of his "*Safe Building.*"

Strength of Pins in Steel Bridge and Roof Trusses.—Iron and steel trusses are now so generally used that it is necessary for the architect who is at all advanced in his profession to know how to determine the strength of the joints and especially of pin joints; and to facilitate the calculation of the necessary size of pins, we give Table V., which shows the single shearing strength and bearing value of pins, and Table VI., showing the maximum bending moment allowed in pins.

Pins must be calculated for shearing, bending, and bearing strains, but one of the latter two only (in almost every case) determines the size to be used.

By bearing strain is meant the force required to crush the edges of the iron plates against which the pin bears.

The several strains per square inch usually allowed on pin connections in bridges are: shearing, 7,500 pounds; crushing, 12,000 pounds; and bending, 15,000 pounds for iron, and 20,000 pounds for steel. In roof trusses, 22,500 lbs. fibre stress is commonly allowed.

The shearing strain is measured on the area of cross-section; the crushing strain, on the area measured by the product of the diameter of the pin by the thickness of the plate or web on which it bears.

The bending moment is determined by the same rules as given for determining the bending moment of beams.

When groups of bars are connected to the same pin, as in the lower chords of trusses, the sizes of bars must be so chosen, and the bars so placed, that at no point on the pin will there be an excessive bending strain, on the presumption that all the bars are strained equally per square inch.

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* Modern Framed Structures, p. 261.

TABLE V.—SHEARING AND BEARING VALUES OF PINS FOR ONE INCH THICKNESS OF PLATE.

Diameter of pin.	Area of pin.	Bearing value at 12,000 lbs. per sq. in.	Single shear 7,500 lbs. per sq. in.	Diameter of pin.	Area of pin.	Bearing value at 12,000 lbs. per sq. in.	Single shear 7,500 lbs. per sq. in.	Diameter of pin.	Area of pin.	Bearing value at 12,000 lbs. per sq. in.	Single shear 7,500 lbs. per sq. in.
Inches.	Sq. in.	Lbs.	Lbs.	Inches.	Sq. in.	Lbs.	Tons.	Inches.	Sq. in.	Lbs.	Tons.
1	.785	12,000	5,890	3	7.069	36,000	26.5	5	19.64	60,000	73.6
$1\frac{1}{8}$	.994	13,500	7,455	$3\frac{1}{8}$	7.670	37,500	28.7	$5\frac{1}{8}$	20.63	61,500	77.3
$1\frac{1}{4}$	1.227	15,000	9,202	$3\frac{1}{4}$	8.296	39,000	31.0	$5\frac{1}{4}$	21.65	63,000	81.2
$1\frac{3}{8}$	1.485	16,500	11,132	$3\frac{3}{8}$	8.946	40,500	33.5	$5\frac{3}{8}$	22.69	64,500	85.1
$1\frac{1}{2}$	1.767	18,000	13,252	$3\frac{1}{2}$	9.621	42,000	36.0	$5\frac{1}{2}$	23.76	66,000	89.1
$1\frac{5}{8}$	2.074	19,500	15,555	$3\frac{5}{8}$	10.32	43,500	38.7	$5\frac{5}{8}$	24.85	67,500	93.2
$1\frac{3}{4}$	2.405	21,000	18,037	$3\frac{3}{4}$	11.05	45,000	41.4	$5\frac{3}{4}$	25.97	69,000	97.3
$1\frac{7}{8}$	2.760	22,500	20,707	$3\frac{7}{8}$	11.79	46,500	44.2	$5\frac{7}{8}$	27.11	70,500	101.1
2	3.142	24,000	23,565	4	12.57	48,000	47.0	6	28.27	72,000	106
$2\frac{1}{8}$	3.547	25,500	26,600	$4\frac{1}{8}$	13.36	49,500	50.1	$6\frac{1}{8}$	29.46	73,500	110
$2\frac{1}{4}$	3.976	27,000	29,820	$4\frac{1}{4}$	14.19	51,000	53.2	$6\frac{1}{4}$	30.68	75,000	115
$2\frac{3}{8}$	4.430	28,500	33,225	$4\frac{3}{8}$	15.03	52,500	56.3	$6\frac{3}{8}$	31.92	76,500	119
$2\frac{1}{2}$	4.909	30,000	36,817	$4\frac{1}{2}$	15.90	54,000	59.6	$6\frac{1}{2}$	33.18	78,000	124
$2\frac{5}{8}$	5.412	31,500	40,590	$4\frac{5}{8}$	16.80	55,500	63.0	$6\frac{5}{8}$	34.47	79,500	129
$2\frac{3}{4}$	5.940	33,000	44,550	$4\frac{3}{4}$	17.72	57,000	66.3	$6\frac{3}{4}$	35.79	81,000	134
$2\frac{7}{8}$	6.492	34,500	48,690	$4\frac{7}{8}$	18.67	58,500	70.0	$6\frac{7}{8}$	37.12	82,500	139

TABLE VI.—MAXIMUM BENDING MOMENTS IN INCH POUNDS TO BE ALLOWED ON PINS FOR MAXIMUM FIBRE STRAINS OF 15,000, 20,000, AND 22,500 POUNDS PER SQUARE INCH.*

Diameter of pin.	Moment for $S=$ 15,000.	Moment for $S=$ 20,000.	Moment for $S=$ 22,500.	Diameter of pin.	Moment for $S=$ 15,000.	Moment for $S=$ 20,000.	Moment for $S=$ 22,500.
In.	Lbs. in.	Lbs. in.	Lbs. in.	Inches	Lbs. in.	Lbs. in.	Lbs. in.
1	1,470	1,960	2,210	4	94,200	125,700	141,400
1⅛	2,100	2,800	3,140	4⅛	103,400	137,800	155,000
1¼	2,880	3,830	4,310	4¼	113,000	150,700	169,600
1⅜	3,830	5,100	5,740	4⅜	123,300	164,400	185,000
1½	4,970	6,630	7,460	4½	134,200	178,900	201,300
1⅝	6,320	8,430	9,480	4⅝	145,700	194,300	218,500
1¾	7,890	10,500	11,800	4¾	157,800	210,400	236,700
1⅞	9,710	12,900	14,600	4⅞	170,600	227,500	255,900
2	11,800	15,700	17,700	5	184,100	245,400	276,100
2⅛	14,100	18,800	21,200	5⅛	198,200	264,300	297,300
2¼	16,800	22,400	25,200	5¼	213,100	284,100	319,600
2⅜	19,700	26,300	29,600	5⅜	228,700	304,900	343,000
2½	23,000	30,700	34,500	5½	245,000	326,700	367,500
2⅝	26,600	35,500	40,000	5⅝	262,100	349,500	393,100
2¾	30,600	40,800	45,900	5¾	280,000	373,300	419,900
2⅞	35,000	46,700	52,500	5⅞	298,600	398,200	447,900
3	39,800	53,000	59,600	6	318,100	424,100	477,100
3⅛	44,900	59,900	67,400	6⅛	338,400	451,200	507,600
3¼	50,600	67,400	75,800	6¼	359,500	479,400	539,300
3⅜	56,600	75,500	84,900	6⅜	381,500	508,700	572,300
3½	63,100	84,200	94,700	6½	404,400	539,200	606,600
3⅝	70,100	93,500	105,200	6⅝	428,200	570,900	642,300
3¾	77,700	103,500	116,500	6¾	452,900	603,900	679,400
3⅞	85 70(	114,200	128,500	6⅞	478,500	638,000	717,800

REMARKS.—The following is the formula for flexure applied to pins:

$$M = \frac{S\pi d^3}{32} \quad \text{or} \quad = \frac{SAd}{8}.$$

M = moment of forces for any section through pin.
S = strain per sq. in. in extreme fibres of pin at that section.
A = area of section.
d = diameter.
π = 3.14159.

The forces are assumed to act in a plane passing through the axis of the pin.

The above table gives the values of M for different diameters of pin, and for three values of S.

If M max. is known, an inspection of the table will therefore show what diameter of pin must be used in order that S may not exceed 15,000, 20,000, or 22,500 lbs., as the requirements of the case may be.

For railroad bridges proportioned to a factor of safety of 5, it is customary to make S max. = 15,000 lbs. in iron and = 20,000 lbs. in steel.

* Carnegie Steel Company's Handbook.

The following example will show the method of determining the size of pin in a simple joint.

EXAMPLE 5.—Determine the size of pin for the joint shown by Fig. 16, which is in the lower chord of a steel truss, the middle bar being a vertical suspension rod merely to hold the chord in place.

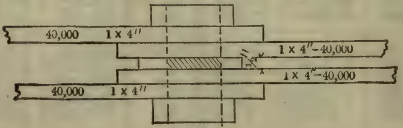

Fig. 16

Ans. The shearing and crushing strain in this case is 40,000 pounds. The bending moment will be the stress multiplied by the distance between the centres of the two outer bars or 40,000 $\times 1'' = 40{,}000$ inch pounds. From Table VI., we find that to sustain a bending moment of 40,000 pounds, with a fibre strain of 20,000 pounds, will require a $2\frac{3}{4}''$ pin. From Table V. we find that the bearing value of a $2\frac{3}{4}''$ pin is but 33,000 pounds, and that we must increase the size of the pin to $3\frac{3}{8}$ inches. The shearing strength of a $3\frac{3}{8}''$ pin is, from Table V., 67,000 pounds, so that the size of pin we must use in this case is determined by the bearing strain. To be sure of the correct size of the pin, one must make the calculation for all three of the strains.

Bending Moment in Pins.

The only difficult part of the process of calculating the sizes of pins will generally be found in determining the bending moment. In cases where the strains all act in the same plane, the bending moment can generally be determined by multiplying the outside force by the distance from its centre to the centre of the next bar, as in the foregoing example. When, however, the forces act in several planes, as is generally the case, the process of determining the bending moment is more difficult, and can be best determined by a graphic process, first published by Prof. Chas. Green, and included in his lectures to the students in engineering at the University of Michigan.

EXAMPLE 6.—As the pieces acting on any well-designed joint are symmetrically arranged, it is unnecessary to consider more than one-half of their number. Fig. 17 shows a sketch of one-half the members of a joint in the lower chord of a Howe truss,

The pieces are parallel to the plane of the paper, and the pin is perpendicular to the same, but drawn in cabinet perspective, at an angle of 45° with a horizontal.

The bars are assumed to be each one inch thick and the channel to have one-half-inch web. The centre of the hanger is $\frac{3}{4}''$ from the centre of the channel.

The method of obtaining the bending moment is as follows:

Draw the line *A B* (Fig. 18) at an angle of 45° with a horizontal, and, commencing with *c*, lay off the distances between the centres of the bars to a scale ($1\frac{1}{2}''$ or $3''$ to the foot will be found most convenient); then draw the lines 1–2, 2–3, etc., parallel to the pieces which they represent in the truss, to a scale of pounds. Resolve the oblique forces into their horizontal and vertical components (in this example there is but one oblique force).

Next draw the stress diagram (Fig. 19) as follows: On a horizontal line lay off 1–2, equal to the first or outer force; 2–3, equal to the next, then 3–4; and 4–1, being the horizontal component of the brace, closes the figure. In the same way, lay off the vertical forces 1–5, 5–6 and 6–1. If the forces are correct, the sum of the forces acting in one direction will always equal those acting in the opposite direction. From 1 draw the line 1–0 at 45°, equal to, say, 20,000 pounds, or any other convenient number at the same scale. Draw 0 2, 0 3, 0 4, etc. Then, in Fig. 18, starting at the first horizontal force, draw *c d* parallel to 0 2, *d e* parallel to 0 3, *e f* parallel to 0 4, and *f k* parallel to 0 1.

In the same way, starting at the first vertical force, draw *r s* parallel to 0 5, *s t* parallel to 0 6, and *t v* parallel to 0 1. Then the line *c d e f k* will represent the boundary of the horizontal ordinates, and *r s t v* the boundary of the vertical ordinates. And to find the resultant of these ordinates at any point on the pin it is only necessary to draw the diagonal from the ends of the ordinates at that point. Thus, the resultant at *X*, Fig. 18, will be *m–n*, and it is evident that this is the longest hypothenuse which can be drawn; and this hypothenuse, multiplied by 0–1 (20,000 pounds) gives 52,500 pounds as the maximum bending moment on the pin.

To obtain the maximum bending moment it is necessary to take the longest hypothenuse that can be drawn, no matter at what place it occurs.

If one desires to try the effect of changing the order of the bars on the pin, it can readily be done. Suppose the diagonal tie to change places with the next chord bar. The horizontal stress

diagram then becomes 1–2, 2–3, 3–4′, 4′–1. The equilibrium polygons will now be (Fig. 20) $c\ d\ e\ f'\ k'$ and $r'\ s'\ t'\ w$ and the

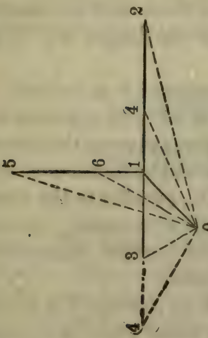

FIG. 19

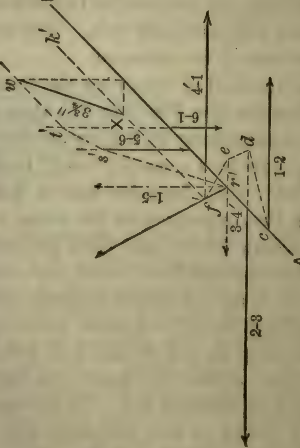

FIG. 20

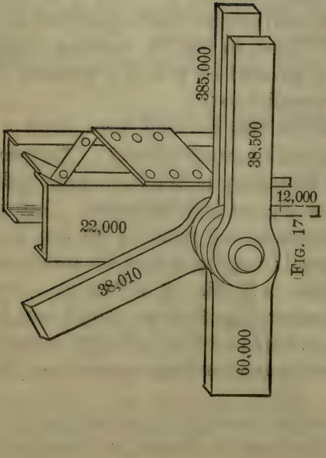

FIG. 17

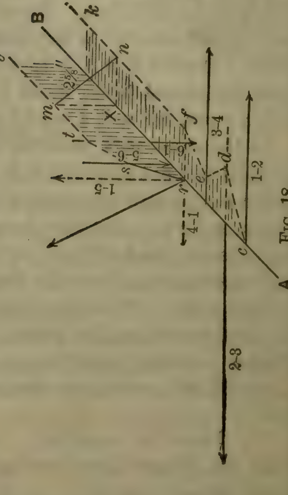

FIG. 18

longest hypothenuse, $w\ x$, or $3\frac{3}{4}''$, which makes the bending moment 75,000 pounds, showing that the arrangement in Fig. 17 is the best.

As a rule, in arranging the bars on a pin, those forces which counteract each other should be close together.

EXAMPLE 7.—To further illustrate this method of determining the bending moment on pins, we will determine the bending

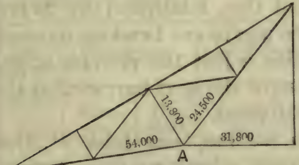

FIG. 21

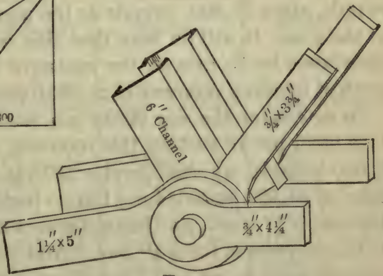

FIG. 22

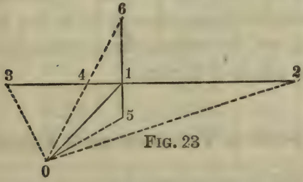

FIG. 23

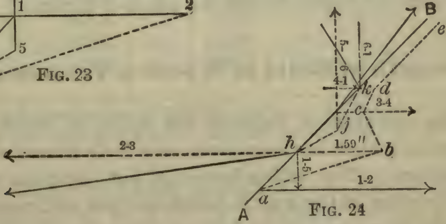

FIG. 24

moment for the pin at the point A, Fig. 21. This is the same truss as worked out in Example 10, Chapter XXVI., the strains given in Fig. 21 being $\frac{1}{2}$ of the strains at the joint, as all the pieces are doubled. Fig. 22 shows the size and arrangement of the ties and strut. It is assumed that the web of the channel is reinforced to make it $\frac{5}{8}''$ thick. Drawing the line AB, Fig. 24, we lay off the outer force at a; then measuring off an inch, the distance between centres of the two outer bars, we lay off the next force parallel to the direction in which it acts; and in the same way, the other two forces. The three inclined forces must be resolved into their horizontal and vertical components. We next draw the stress diagram (Fig. 23) to the same scale of pounds, making 1 0 equal to 20,000 pounds. The lines 0 4 and 0 6 happen, in this case, to coincide. Then, in Fig. 24, we draw $a\ b$ parallel to 0 2, $b\ c$ parallel to 0 3, $c\ d = 0\ 4$, and $d\ e$ parallel to 0 1. In

the same way, we obtain the line $h\ j\ k\ B$. In this case, it will be seen that the longest horizontal ordinate is $h\ b$, while at that point there is no vertical ordinate; also, that no hypothenuse can be drawn which will be as long as $h\ b$, so that we must take $h\ b$ as the greatest resultant; and this, multiplied by 20,000 pounds, gives 31,800 pounds as the maximum bending moment on the pin. It wlll be seen that this is just the product of the outer force by its arm to the centre of the next bar, so that the greatest bending moment is at that point.

To determine the size of the pin, we find, from Table VI., that for a steel pin to sustain this moment, allowing a fibre strain of 20,000 pounds, we shall need a $2\frac{5}{8}''$ pin. This pin has a bearing value of 31,500 pounds for a bar an inch thick. The outer bar in this case is $\frac{3}{4}''$ thick, and has a strain of 31,800 pounds, equivalent to 42,400 pounds for a $1''$ bar. And we see, from Table V., that we shall need to use a $3\frac{1}{2}''$ pin to meet this strain. The shearing strength of a $3\frac{1}{2}''$ pin is 36 tons, or more than double the strain. Hence we must use a $3\frac{1}{2}''$ pin, or, by increasing the thickness of the bars, we might reduce the pin to 3 inches.

Strength of Bolts in Wooden Trusses and Girders.

The strength of bolts in the joints of wooden structures is something that has been given very little attention by writers and engineers. The author knows of but one book which gives any explanation of the way in which the size and number of the bolts should be computed, or how to determine the stresses in them.* The only actual knowledge that we have of the strength of bolt joints in timber trusses is that derived from a series of tests made at the Massachusetts Institute of Technology and described in the *Technology Quarterly* for September, 1897. The results of these tests were discussed by the author in the *Engineering Record* of November 17, 1900.

The following rules and tables are entirely original with the author; they are based upon the general principles of stresses, upon the tests above referred to, and upon a very extended experience with wooden trusses. The author believes that when correctly used they will give perfectly safe results without an undue excess of strength.

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* The Design of Simple Roof Trusses in Wood and Steel by M. A. Howe, C.E.—John Wiley & Sons, Publishers.

TABLE VII.—PERMISSIBLE BEARING VALUE (END COMPRESSION) OF BOLTS IN TIMBER, PER INCH OF LENGTH, AND DISTANCE CENTRE OF BOLT MUST BE FROM END OF TIMBER OR FROM THE NEXT BOLT.*

Diameter of bolt in ins.	Yellow pine.		Oregon pine or oak.		Spruce.		White or soft pine.	
	Bearing, lbs.	Distance, ins.	Bearing, lbs.	Distance, ins.	Bearing, lbs.	Distance, ins.	Bearing, lbs.	Distance, ins.
$\frac{3}{4}$	1125	5	1012	$4\frac{1}{2}$	900	5	750	5
$\frac{7}{8}$	1310	6	1183	5	1050	$5\frac{1}{2}$	880	$5\frac{1}{2}$
1	1500	6	1350	$5\frac{1}{2}$	1200	$6\frac{1}{2}$	1000	6
$1\frac{1}{8}$	1690	7	1520	6	1350	7	1125	7
$1\frac{1}{4}$	1870	$7\frac{1}{2}$	1690	$6\frac{1}{2}$	1500	$7\frac{1}{2}$	1250	8
$1\frac{3}{8}$	2060	8	1860	$7\frac{1}{2}$	1650	8	1375	9
$1\frac{1}{2}$	2250	9	2025	8	1800	9	1500	10
$1\frac{3}{4}$	2625	10	2360	9	2100	10	1750	11
2	3000	11	2700	10	2400	12	2000	12
$2\frac{1}{4}$	3375	12	3040	11	2700	$13\frac{1}{2}$	2250	$13\frac{1}{2}$
$2\frac{1}{2}$	3750	13	3375	12	3000	15	2500	15
$2\frac{3}{4}$	4125	14	3710	13	3300	$16\frac{1}{2}$	2750	$16\frac{1}{2}$
3	4500	15	4050	14	3600	18	3000	18

* Based on a bearing resistance of 1500 lbs. per sq. in. for Southern yellow pine, 1350 lbs. for oak and Oregon pine, 1200 lbs. for spruce, and 1000 lbs. for white pine.

TABLE VIII.—PERMISSIBLE BEARING VALUE (ACROSS THE GRAIN) OF BOLTS IN TIMBER, PER INCH OF LENGTH.*

Diameter of bolt in ins.	Yellow pine.	Oregon pine.	Spruce.	White pine.	Oak.
$\frac{3}{4}$	375	300	225	187	450
$\frac{7}{8}$	437	350	262	220	525
1	500	400	300	250	600
$1\frac{1}{8}$	562	450	337	280	675
$1\frac{1}{4}$	625	500	375	310	750
$1\frac{3}{8}$	687	550	412	345	825
$1\frac{1}{2}$	750	600	450	375	900
$1\frac{3}{4}$	875	700	490	437	1000
2	1000	800	600	500	1200

* Based on unit stresses of 600 lbs. per sq. in. for oak, 500 for Southern yellow pine, 400 for Oregon pine, 300 for spruce, and 250 for white pine.

TABLE IX.—MAXIMUM PERMISSIBLE TENSION, SHEAR, AND BENDING MOMENT (IN INCH-POUNDS) FOR WROUGHT-IRON BOLTS IN TIMBER.

Diam. of bolt in ins.	Tension, lbs.*	Single shear, lbs.	Bending moment, inch-lbs.	Diam. of bolt in ins.	Tension, lbs.*	Single shear, lbs.	Bending moment, inch-lbs.
$\frac{3}{4}$	6,000	4,420	931	$1\frac{3}{4}$	35,000	24,050	11,800
$\frac{7}{8}$	8,400	6,010	1,479	2	45,000	31,416	17,700
1	10,860	7,850	2,210	$2\frac{1}{4}$	60,000	39,760	25,200
$1\frac{1}{8}$	13,720	9,940	3,140	$2\frac{1}{2}$	74,000	49,080	34,500
$1\frac{1}{4}$	17,700	12,270	4,310	$2\frac{3}{4}$	92,000	59,400	45,900
$1\frac{3}{8}$	21,400	14,480	5,740	3	108,000	70,700	59,600
$1\frac{1}{2}$	25,740	17,670	7,460	$3\frac{1}{4}$	130,000	82,950	75,800

Unit stresses: Tension, 20,000 lbs., shear, 10,000 lbs., bending 22,500 lbs.
* To be used only in truss joints. Not safe for rods.

General Principles.—As a rule, bolts as used in wooden trusses and girders are subject to the same kind of stresses as pins or rivets in steel structures, although occasionally they are subject only to direct tension. *When the pieces joined are not more than two inches thick,* so that they can be drawn tightly together, thereby producing a good deal of resistance from friction, the bolts may be considered as rivets and proportioned for shearing and bearing only, the bending moment being neglected.

When the pieces of wood joined are *more* than two inches thick, the bolts should be proportioned for shearing, bearing, and flexure.

Tables.—To facilitate computing the number and size of the bolts, the author has prepared Tables VII., VIII., and IX., giving the safe resistance of bolts of different sizes to each kind of stress, and also the length of wood required beyond or between the bolts to prevent shearing of the wood.

The bearing resistance is determined by the resistance of the wood to crushing, and not by the resistance of the bolt, but for convenience it is considered as a property of the bolt. The resistance of wood to crushing across the grain being very much less than against end wood, the bearing resistance both ways is given. With these tables it is only necessary to compute the stress of each kind, and to select the bolt or number of bolts that will resist all of the stresses.

The unit stresses allowed in computing the tables are somewhat larger than are recommended for struts or rods, but for the conditions under which bolts are generally used the author believes the tables may be used with perfect confidence. The *safe tension* given in Table IX., however, should only be used when the stress is computed as directed in Case V.

Special Cases.—The various ways in which bolts are used in wooden structures to transmit a load or stress from one piece to another are illustrated by Figs. 25 to 35, which may be divided into five cases, each of which, to make the explanation more readily understood, is treated separately.

Case 1. *Bolts in Built-up Tie-beams.*—Figs. 25 and 26. Tie-beams of wooden trusses, when more than 30 feet long, must usually be built up of several pieces on account of the expense of a single stick of timber of the full length. Such tie-beams can generally be built to the best advantage of 2-inch plank, breaking joint and bolted together side by side.

As the tensile stress in a tie-beam is often very considerable, the placing of the joints and the size and number of the bolts must be determined with much care, otherwise the beam may be pulled apart longitudinally so as to permit the truss to sag, or possibly to collapse.

EXAMPLE 8.—Fig. 25 is given as a typical example of a built tie-beam. It represents the tie-beam of a six-panel Howe Truss of 50 feet span carrying two floors and a roof, the direct tension in the different panels being as indicated on the lower line. The thickness of the planks are drawn out of proportion to the length in order to more clearly show the end-joints. The black circles in the centre plank represent the vertical rods. The wood is to be Oregon pine.

It will be seen that the rods practically cut the centre plank in two, and for this reason it is better to build the tie-beam of an uneven number of planks rather than of an even number, and the centre course of planks should be jointed at the rods.

No dependence is placed upon the centre course of planks for resisting the tension, although they count in resisting the transverse strain.

The length of the planks that should be used for the outer courses, and the way in which they should break joint, is a matter that will vary for different trusses, and for which no definite rule can be given.

In general the planks should be of such length and so arranged

that the distance between the joints in adjacent courses, as at X and Y, will be as great as practicable, and so that not more than two joints will come opposite each other.

For the tie-beam in question, the length and arrangement shown is as good as can be devised. This gives a distance between X and Y of 12 feet; which is about as little as would answer. At the centre, where the stress is greatest, only one joint occurs at any given cross section.

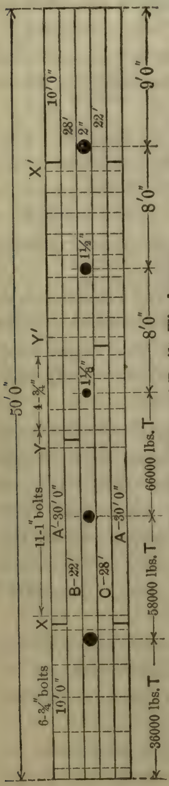


Fig. 25. Plan of Built Tie-beam.

In this beam, the two outer planks $A'A$ must be capable of resisting the full tensile strength in the centre panels of the truss, and the planks B and C must transmit the stress in the second panel to the end braces. We must therefore place enough bolts between the joints X-Y to transmit the stress from the outer planks to planks B and C.

The stress to be transmitted is the stress in the second panel, or 58,000 lbs. We may assume that A' will receive one half of this, or 29,000 lbs., and that if we provide enough bolts to transmit 29,000 lbs. from A' to B, the other side of the tie-beam will carry a like strain.

These bolts must be computed for shearing and bearing. The bolts will be in single shear between A' and B. We will now see how many 1-inch bolts will be required to resist a shearing and bearing stress of 29,000 lbs.

From Table IX. we find the resistance of a 1-inch bolt to single shear to be 7,850 lbs.; hence, to resist a stress of 29,000 lbs. will require four bolts.

The resistance of a 1-inch bolt in Oregon pine (Table VII.) is 1,350 lbs. per inch of bearing. As the bearing in each plank is 2 inches, we have 2,700 lbs. as the safe resistance to bearing of one bolt, consequently it will require eleven 1-inch bolts to avoid crushing the wood. As the number is larger than that required to resist shearing, it is the number of bolts required.

These bolts must be placed between the points X and Y at each side of the centre. From the distance column in Table VII. we see that the bolts should be placed at least $5\frac{1}{2}$ inches from the end of the planks. Calling it 6 inches and putting two bolts at X and two at Y, we must divide the remaining space, 11 feet, by the remaining 7 bolts. As there will be 8 spaces, the bolts will be $16\frac{1}{2}$ inches apart, longitudinally. They should be staggered, as shown in Fig. 26.

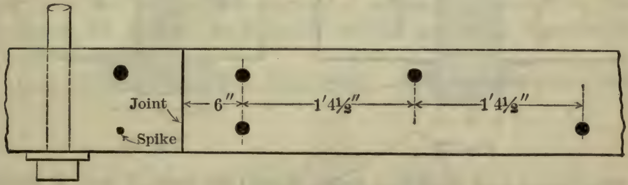


Fig. 26.—Elevation of Beam opposite X.

These 11 bolts each side of the centre will be sufficient to transmit the entire tensile stress to the supports, but bolts should also be inserted between the points Y–Y', and between X–X' and the ends, to bind the planks together. These bolts, however, need not be as large, nor as near together; $\frac{3}{4}$-inch bolts will answer, spaced about 2 feet on centres. Two bolts should always be placed at the end of a built tie-beam.

The bolts should be of wrought iron or mild steel, driven through holes of the same size as the bolt, provided with washers, and the nuts screwed up tight.

Case II. Bolts in Girders, as in Figs. 27 *and* 28.—The construction shown by Figs. 27 and 28 is very commonly used, where a girder cannot project its entire depth below the floor joists. The bolts in Fig. 27 should be computed for resistance to bearing and shearing, and in Fig. 28 to bearing, shearing, and flexure. In either case *the resistance to single shear* must equal S or S', whichever is the larger.

$$\text{Bearing on wood per inch} = \frac{S+S'}{B}.$$

In case Fig. 28, Bending moment $= \frac{S\,L}{2}$ or $\frac{S'\,L}{2}$, whichever is larger, B and L being measured in inches.

EXAMPLE 9.—We will suppose that we wish to use the construction shown in Fig. 27. The girder to be 8″×14″, with a span of 14 ft. The floor joists to be 3″×12″, with a span of 20 ft., measured from the centres of girders. Joists and girders to be of Oregon pine. Angles to be 4″×3½″×⅜″. The floor load to be figured at 60 lbs. per sq. ft., including weight of floor.

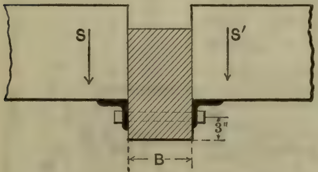


Fig. 27

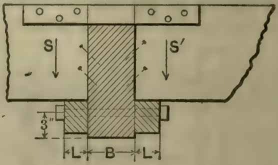


Fig. 28

How many and what size bolts should be used, S and S' being equal?

Ans. The floor area supported by girder = 14 × 20 = 280 sq. ft. at 60 lbs. per sq. ft. $S+S' = 280 \times 60 = 16{,}800$ lbs. or $S = 8{,}400$ lbs. Try ¾″ bolts. Total single shear = 8,400 lbs. Resistance of one ¾″ bolt to single shear, (see Table IX.) = 4,420 lbs. Hence two bolts will resist the shearing stress.

$$\text{Bearing stress per inch} = \frac{16{,}800}{B} = \frac{16{,}800}{8} = 2{,}100 \text{ lbs.}$$

Bearing resistance of one ¾″ bolt in Oregon pine, across the grain, Table VIII, = 300 lbs. Hence seven bolts will be required to prevent crushing of the wood. As the span is 14 ft., this will require a ¾-inch bolt every 2 feet. The centre of the bolts should be at least 3″ above the bottom of the girder.

EXAMPLE 10.—Construction as shown in Fig. 28. Girder 6″×14″, Oregon pine, 12 ft. span. Joists 2″×12″, 18 ft. span to centre of girder, on each side. Total floor load 65 lbs. per sq. ft. Strips on sides of girder 3″×4″; $L = 3″$. How many and what size bolts should be used?

Ans. Load supported by girder = 12′ × 18′ × 65 = 14,040 lbs. $S = 7{,}020$ lbs. = shearing stress.

$$\text{Bearing stress per inch} = \frac{14{,}040}{6} = 2{,}340 \text{ lbs.}$$

$$\text{Bending moment} = \frac{7{,}020 \times 3}{2} = 10{,}530 \text{ lbs.}$$

To resist the shearing stress will require two $\frac{3}{4}''$ bolts. To resist the bearing stress (use Table VIII.) will require eight $\frac{3}{4}''$ bolts, or seven $\frac{7}{8}''$ bolts.

To resist the bending moment (use Table IX.) will require eleven $\frac{3}{4}''$ bolts, or seven $\frac{7}{8}''$ bolts. Eleven $\frac{3}{4}''$ bolts would give a spacing of only about 13″, allowing for one bolt at each end, and this would injure the girder, hence we must use seven $\frac{7}{8}''$ bolts, which would be spaced about 22″ on centres. Practically the strain on the bolts will be somewhat relieved by toe-nailing the joists to the girder, but it is not safe to put much dependence upon the spikes.

Case III. Pin Bolts, as in Figs. 29–31.—Whenever ties or struts

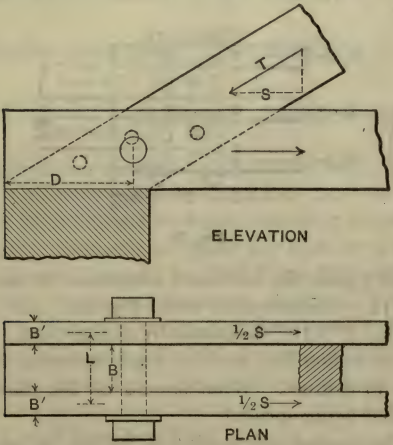


Fig. 29

are joined by bolts in the manner indicated by Figs. 29–31, and the thickness *B* exceeds 2 inches, the diameter of the bolt or

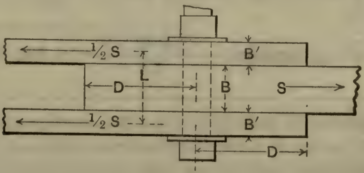


Fig. 30

bolts should be computed for resistance to *shearing, bearing* and *flexure.*

For any one of these joints the stresses will be as follows:

$$\text{Single shear} = \frac{s}{2}.$$

$$\text{Bearing pressure on pin per inch of length} = \frac{S}{B}.^*$$

$$\text{Bending moment}\dagger = \frac{S \times L}{12}.$$

If there should be more than one bolt, divide the stress obtained by the above formulas by the number of bolts, to find the stress in each bolt.

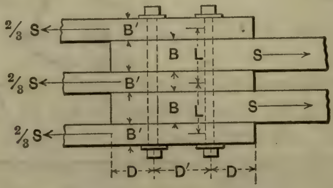


Fig. 31

In Fig. 29 S equals the *horizontal component* of the thrust T.

Example 11.—Construction as in Fig. 29. A $6'' \times 10''$ rafter is joined to two $3'' \times 10''$ tie-beams by a single bolt. The thrust in the rafter is 30,000 lbs., and the angle between the rafter and tie-beams is 30 degrees. What should be the diameter of the bolt, the timber being spruce?

Ans. The horizontal component of 30,000 lbs. at an angle of 30° is practically 26,000 lbs. Then $S=26{,}000$, $B=6$ inches, $L=9$ inches. Single shear $=13{,}000$ lbs.

$$\text{Bearing pressure per inch of bolt} = \frac{26{,}000}{6} = 4{,}33\dot{3} \text{ lbs.}$$

$$\text{Bending moment} = \frac{26{,}000 \times 9}{12} = 19{,}500 \text{ lbs.}$$

---

* This holds good only when B' in Figs. 29 and 30 is one half or more of B, and for Fig. 31, when B' is two thirds or more of B.

† Based on the supposition that the bolt acts as a beam *fixed at both ends* and uniformly loaded.

From Table IX. we find that to resist a shearing stress of 13,000 lbs. will require a $1\frac{5}{8}''$ bolt, and to resist a bending moment of 19,500 lbs. will require a $2\frac{1}{4}$-inch bolt.

To resist a bearing pressure of 4,333 lbs. on end wood in spruce will require a larger bolt than is given in the table (Table VII.). If we divide our stress of 4,333 lbs. by 1,200, the allowed pressure per sq. inch for spruce, we obtain $3\frac{5}{8}$ inches for the diameter of the bolt; hence in this example the diameter of the bolt is determined by the bearing pressure. This is a larger bolt than it is desirable to use, and we will see what diameter will be required if we make the strut $8''\times 8''$, and the tie-beams $4''\times 8''$, as the strength of the timber would be about the same.

Using these dimensions, $B=8$ ins. and L, 12 ins. This will give us a bearing pressure of $\frac{26,000}{8}=3,250$ lbs. and a bending moment of $\frac{26,000\times 12}{12}=26,000$ lbs. The shear will be the same as before, as it is not affected by the width of B.

To resist a bearing pressure of 3,250 lbs. will require (see Table VII.) a $2\frac{3}{4}''$ bolt, and to resist a bending moment of 26,000 lbs., a $2\frac{1}{2}''$ bolt. Hence if we make the strut $8''\times 8''$ we should use a $2\frac{3}{4}''$ bolt, and D should be $16\frac{1}{2}''$ (from Table VII.).

Example 12.—Same construction as above, but with three bolts, placed as shown by dotted circles, instead of one. Strut to be $6''\times 10''$; tie-beams, $3''\times 10''$, spruce. Stress to be the same as in the above example. The shearing, bearing, and bending stresses will also be the same, but as we are using three bolts, each stress should be divided by three.

This will give the stress on each bolt, and the corresponding size of bolt, as follows:

Shearing stress	=4,333 lbs.,	requires a	$\frac{7}{8}$-inch bolt.
Bearing pressure	=1,444 lbs.,	" "	$1\frac{1}{4}$-inch bolt.
Bending moment	=6,500 lbs.,	" "	$1\frac{1}{2}$-inch bolt.

In this case the bending moment requires the largest bolt, and we must use three $1\frac{1}{2}$-inch bolts.

Example 13.—Construction as in Fig. 30. Centre beam to be $6''\times 8''$, and outer beams $3''\times 8''$, all of Oregon pine.

Assume tension in centre beam to be 24,000 lbs. What should be the diameter of the bolt?

Ans. $S=24,000$; $B=6''$, and $L=9''$.

Single shear $=12,000$ lbs.

$$\text{Bearing pressure per inch} = \frac{24,000}{6} = 4,000 \text{ lbs.}$$

$$\text{Bending moment} = \frac{24,000 \times 9}{12} = 18,000 \text{ lbs.}$$

To resist the shear will require a $1\frac{1}{4}''$ bolt, to resist bearing pressure a $3''$ bolt, and to resist the bending moment will require a $2\frac{1}{4}''$ bolt. For a $3''$ bolt the distance D should be $14''$.

Example 14.—Same conditions as in Example 13, except that two bolts, one behind the other, will be used. With two bolts, each stress will be one-half of that for a single bolt. Dividing the stresses obtained in example 13 by 2, we have:

Single shear $=6,000$ lbs., requires a $\frac{7}{8}$-inch bolt.
Bearing pressure $=2,000$ lbs., requires a $1\frac{1}{2}$-inch bolt.
Bending moment $=9,000$ lbs., requires a $1\frac{5}{8}$-inch bolt.

Hence the size of the bolt is determined by the bending moment, and we must use two $1\frac{5}{8}$-inch bolts. For a $1\frac{5}{8}$-inch bolt D should be at least $8\frac{1}{2}$ inches, and the distance between the bolts an inch or so more, say 10 inches.

Case IV. Strop and Bolt Joint, as in Fig. 32.—The construction indicated by Fig. 32 is sometimes used to secure the foot of

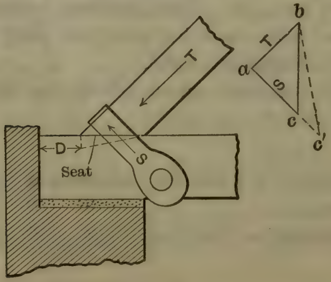


Fig. 32

the rafter in wooden trusses to the tie-beam. When the distance D is sufficient to resist the shearing stress, as explained at the beginning of this chapter, the strap is only of value in

holding the rafter in place, and the bolt is not subject to any great stress. When it is impossible to get the necessary length for D, then the strap and bolt should be computed to resist the full stress.

As the strap should not usually be more than ½″ to ¾″ thick, only the shear and bearing pressure on the bolt need be considered. These should be computed as follows:

$$\text{Single shear} = \frac{S}{2} = \text{tension in strap.}$$

Bearing pressure per inch on wood $= \frac{S}{B}$ where B = breadth of tie-beam in inches.

Bearing pressure per inch on strap $= \frac{S}{2t}$, where t equals thickness of strap in inches.

To find the value of S, draw a line T, representing the thrust in the rafter to a scale of pounds to the inch, and parallel with the axis of the rafter. From the end a draw an indefinite line parallel to the axis of the strap, and from the end b a line at *right angles* to the *seat* of the rafter. These lines will intersect at C, and ac, measured by the scale used in drawing T, will give the value of S in pounds. If the rafter in Fig. 32 rests on the top of the tie-beam, then bc will be vertical, but if the tie-beam is notched out, as shown by the dotted lines, then the line from b should be drawn *at right angles to the bottom of the notch,* which will give the point c'. It will be seen that notching the tie-beam increases the stress in the strap.

EXAMPLE 15.—In a king rod truss of 36 feet span the compression in the rafter is 18,000 lbs., and the inclination of the rafter 45°. The rafter is to be 6″×6″ and the tie-beam 6″×8″, both of spruce. What size strap and pin-bolt will be required to hold the foot of the rafter, without notching into the tie-beam?

Ans. The first step is to determine the stress S. As the inclination of the rafter is 45°, and the seat of the rafter is horizontal, the line ac (Fig. 32) must equal ab; hence S will equal T, or 18,000 lbs.

Then the single shear on bolt will be 9,000 lbs.
Tension in each side of strap will be 9,000 lbs.

$$\text{Bearing pressure per inch on wood} = \frac{18{,}000}{6} = 3{,}000 \text{ lbs.}$$

$$\text{Bearing pressure per inch on strap, } \frac{9{,}000}{t}.$$

That the strap may not crush the top of the rafter, it should be at least 3 inches wide. At 10,000 lbs. per sq. inch the sectional area required in the strap will be .9 inch. If we divide this by the width (3) we have .3 inch for the thickness. We will therefore make the strap $\frac{3}{8}$ inch thick and 3 inches wide. The bearing pressure *per inch* on the strap will then be

$$\frac{9,000}{\frac{3}{8}}=\frac{8}{3}\times 9,000=24,000 \text{ lbs.}$$

The bolt must therefore be able to resist a single shear of 9,000 lbs., a bearing pressure against spruce of 3,000 lbs. per inch, and a bearing pressure against the strap of 24,000 lbs, From Table IX. we find that it will require a $1\frac{1}{8}$-inch bolt to resist the shear; from Table VII we find that it will require a $2\frac{1}{2}$-inch bolt to reduce the pressure on the wood to the proper limit, and from Table V. of this chapter that we should use a 2-inch pin for a bearing of 24,000 lbs. on the strap. The largest bolt is that required for the bearing against the wood, or $2\frac{1}{2}$ inches. If the width of the beam and rafter was increased to

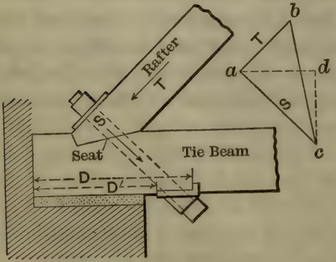


Fig. 33

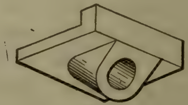


Fig. 34

8 inches, the bearing on the wood per inch would be reduced to 2,250 lbs., which would require only a 2-inch bolt; the same diameter as required for bearing on the strap. As a rule this form of joint is not desirable for trusses having a span of more than 36 feet, or where the compression in the strut exceeds 18,000 lbs. It has the advantage over the joint shown in Fig. 33, however, in that there is no iron work to project below the bottom of the tie-beam.

In the joint (Fig. 33) the bolt is subject to direct tension only, the stress in the bolt being denoted by S. The value of S is found in exactly the same way as explained under Case IV. The rafter may be let into the tie-beam, or it may merely rest on top, the stress in the bolt being least in the latter case, but it is easier to fit the members of the truss together if the rafter is let into the tie-beam say $1\frac{1}{4}$ or $1\frac{1}{2}$ inches. If the shoulder is 4 to 6 inches long it will hold the rafter while the pieces are being fitted together, and after they are fitted, the hole for the bolt can be bored in the right position.

Whenever S exceeds 10,000 lbs. a cast plate, as shown in Fig. 34, made to fit the inclination of the bolt, should be let into the bottom of the tie-beam for the head of the bolt to bear against. *The hole for the bolt* should be $\frac{1}{8}$ inch larger than the diameter of the bolt. *The horizontal component* of S should be determined, and the distances D and D' made sufficient to resist longitudinal shearing.

The horizontal component is found by drawing a vertical line from c and a horizontal line from a, in the diagram, Fig. 33, intersecting at d. ad, measured by the scale of the diagram, will give the horizontal component. The distance $D+D'$ should equal the horizontal component divided by the breadth of the tie-beam multiplied by the value of F, Table I. of this chapter.

EXAMPLE 16.—Conditions the same as in Example 15. Determine the diameter of the bolt, and least distance for D. In this example S will be greater than in Example 15, because the seat of the rafter is not horizontal. Therefore draw $T=18{,}000$ lbs. to a scale, and parallel to the axis of the rafter. (See diagram Fig. 33.) From the lower end of T draw a line parallel to the bolt, and from b a line at right angles to the seat of the rafter. These two lines will meet at c, and ac will give the value of S, which we find to be 27,000 lbs. From Table IX. we find that it will require a $1\frac{5}{8}$-inch bolt to resist a direct tension of 27,000 lbs.; therefore the bolt must be $1\frac{5}{8}$ inches in diameter. On the stress diagram draw a vertical line through c and a horizontal line through a, then ad represents the shearing force to be resisted at D. The line ad we find measures 19,000 lbs. The breadth of the tie-beam is 6 inches, and F, for spruce, Table I., with the grain, is 90 lbs.; then $D+D'$ must $=\dfrac{19{,}000}{6\times 90}$ or 35 inches. In Fig. 33, $D=20$ inches, and D' $15\frac{1}{2}$ inches; therefore, the distance is sufficient.

NOTE.—The author believes that for computing the resistance to longitudinal shearing, in a case like this, where there is a heavy compression on the wood across the grain, it will be perfectly safe to use values for F double those given in Table I. This opinion is based on the tests made at the Massachusetts Institute of Technology, and mentioned on page 382.

EXAMPLE 17.—To determine the size of bolts for the joint shown in Fig. 35, the thrust in the rafter being 65,500 lbs. and the timber being yellow pine.

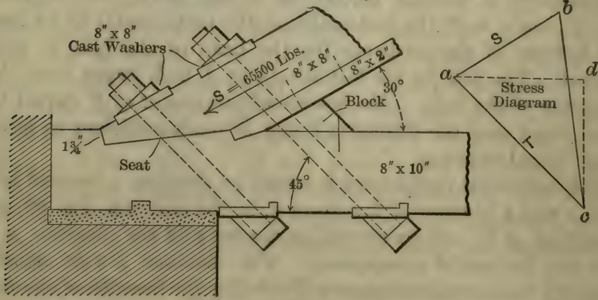


Fig. 35

Ans. The first step is to determine the tension in the rods. This is done by drawing the diagram, commencing with the line ab, which represents the thrust in the rafter. ac is drawn parallel to the bolts and $b\ c$ at right angles to the seat of the rafter. The line ac scales 96,500 lbs., and assuming that the strain will be equal in the two bolts, the tension in each bolt will be 48,250 lbs. From Table IX. we find that this will require a 2⅛-inch bolt. Therefore we must use two bolts of 2⅛ inches diameter.

The horizontal component of ac is represented by the line ad, which scales 68,350 lbs. This will require a shearing area in hard pine of $\frac{68,350}{125}$ or 547 sq. inches. As the tie-beam is 8 inches wide the length must be $\frac{547}{8}$ or 68½ inches. In the drawing we have much more than this, hence there will be no danger of the bottom plates shearing the wood.

Theoretically, the size of the washers should be equal to the stress in one bolt divided by the resistance of the wood to crushing across the grain,* or $\frac{48,250}{500}$ or 96 sq. inches, but as a slight crushing of the fibres in this case would do no particular harm, we will reduce the area to 64 sq. inches or 8×8 inches.

* For resistance of woods to crushing across the grain, see Chapter XIV.

CHAPTER XIII.

PROPORTIONS OF CAST-IRON AND STEEL BEARING-PLATES FOR COLUMNS, BEAMS AND GIRDERS, AND FOR BRACKETS ON CAST COLUMNS.

If a heavily loaded column or girder should rest directly upon a wall or pier of masonry, the weight would be distributed over such a small area that in most cases there would be danger of crushing the masonry, particularly if it were of brick or rubble work. To prevent this a bearing-plate should be placed between

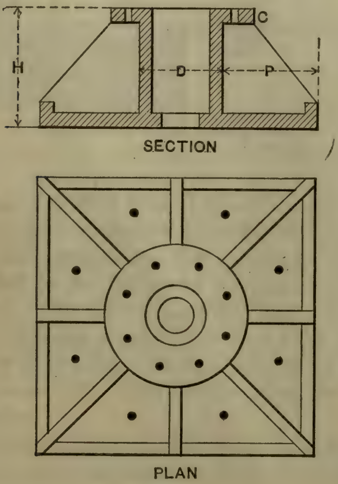


Fig. 1

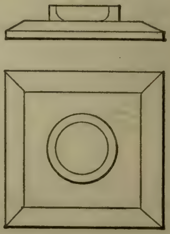


Fig. 2

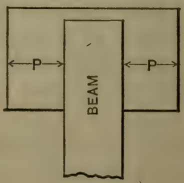


Fig. 3

the end of the beam or column and the masonry, the size of the plate being such that the load from the column or girder divided by the area of the plate shall not exceed the safe crushing strength of the masonry per unit of measurement.

TABLE I.—MAXIMUM LOAD PER SQUARE INCH ON DIFFERENT KINDS OF MASONRY FROM BEARING PLATES.

For granite	1,000 lbs. per sq. in.
" best grades of sandstone	700 " " " "
" soft sandstone	400 " " " "
" hard stone rubble	150 to 250 " " " "
" extra-hard brickwork in cement mortar	150 to 200 " " " "
" good hard Eastern brickwork in lime mortar	120 " " " "
" common brickwork	100 " " " "
" good Portland cement concrete	200 " " " "
" sand or gravel	60 " " " "

EXAMPLE 1.—The basement columns of a six-story warehouse support a possible load of 212,000 pounds each; under the column is a base-plate of cast iron resting on a bed of Portland-cement concrete two feet thick. What should be the dimensions of the base-plate?

Answer.—As the plate rests on concrete, the bottom of the plate should have an area equal to $212{,}000 \div 200 = 1060$ square inches, or 33 inches square. The column should be about 10 inches in diameter and 1 inch thick. The shape of the base-plate should be as shown in Fig. 1.

Shape of Column Base-Plates.—For small columns and wooden posts with light loads, plain flat plates of cast iron are generally used. They may have a raised ring or cross to fit inside the base of a hollow column, or for a wooden post a raised dowel, 1½ inches or 2 inches in diameter. If the plate is very thick, a saving in the weight of the plate may be made by bevelling the edge, as shown in Fig. 2, without loss of strength. The outer edge, however, should not be less than ⅞ inch thick.

When the bearing-plate is so large that the projection beyond the column is more than six inches, a ribbed plate should be used similar to that shown in Fig. 1, which is drawn for a round column. Fig. 9, Chapter XIV., shows a similar base-plate for an H-shaped column. With such plates no transverse strain is developed, and if the column is bolted to the plate, it adds greatly to the stability of the column.

For base-plates similar to Fig. 1 the height H should be equal to the projection P and D should be equal to the diameter of

the column. The thickness of all portions of the plate should be equal, or nearly so, to that of the column above the base. This is not so much required for strength as to get a perfect casting, as such castings are liable to crack by unequal cooling when the parts are of different thickness. The projection of the flange C should be at least 3 inches, to permit of bolting the column to the plate.

For steel columns, base-plates of steel, such as are shown in connection with the details of Z-bar and channel columns, Chapter XIV, are commonly used, although for very heavy steel columns cast-iron base-plates are also used, and where the cast iron is entirely in compression, they are to be preferred to steel bearing-plates.

Calculations for Bearing-Plates.—For ribbed or bracketed plates, such as Fig. 1, proportioned as above described, no other calculation is necessary than that of finding the area of the base, as illustrated by Example 1. With flat plates, however, a transverse strain is developed in the metal, and it is necessary to compute the *thickness* of the plate as well as its size. *To find the size and thickness of flat plates* under columns and posts—

First determine the size of the plate by dividing the load on the column in pounds by the safe resistance of the material on which the plate rests, as explained in Example 1.

Second.—Knowing the size of the plate and the size of the column, determine the projection of the plate beyond the column.

Let w = pressure under the plate in lbs. per sq. in.
W = Load on column in pounds;
A = Area of plate in sq. ins.;
B = one side of square plate in inches;
D = diameter of round column or side of square post in inches:
A' = difference between area of plate and sectional area of column;
P = projection of edge of plate beyond column in inches;
t = thickness of plate in inches;

then

$$A=\frac{W}{w},\quad B=\sqrt{A},\quad \text{and}\quad P=\frac{B-D}{2}.$$

For cast-iron plates,

$$t=\sqrt{\frac{w.P.A.'}{D}}\ \text{divided by } 80. \tag{1}$$

For steel plates,

$$t=\sqrt{\frac{w.P.A.'}{D}}\text{ divided by 220.}$$

EXAMPLE 2.—A yellow-pine post 12 ins. square supports a probable load of 115,200 lbs. The post will rest on a cast-iron plate bedded on first-class brick work in cement mortar. What should be the size and thickness of the plate?

Ans. $W=115{,}200.$ $w=200.$ $A=115{,}200\div200=576$ sq. ins.

$$B=\sqrt{576}=24\text{ ins.}\quad A'=576-144=432\text{ sq. ins.}$$

$$P=\frac{24-12}{2}=6\text{ ins.}$$

$$\text{Then } t=\sqrt{\frac{200\times6\times432}{12}}\div80=\frac{208}{80}=2.6\text{ ins.}$$

For a steel plate $t=\frac{208}{220}$ or 1 inch.

The cast-iron plate may be made 2.6 ins. thick under the post and bevelled to $1\frac{1}{4}$ ins. at the edges.

For a rectangular post the plate should be proportioned so that the projection will be the same on each side of the post.

When computing the area of bearing-plates under columns the *probable* load on the column rather than the *possible* load should be taken.

Bearing-plates Under Beams or Girders.

The ends of heavily loaded beams or girders should rest on bearing-plates, either of iron, steel, or strong smooth stone.

The *area* of these bearing-plates should be computed in the same way as the area of bearing-plates under columns.

The thickness of *cast-iron* plates may be computed by the following formula:

$$t=.024\,P\sqrt{w}.$$

For steel plates

$$t=.0137\,P\sqrt{w}$$

in which w = the safe bearing resistance of the masonry per sq. inch, and P equals the projection of the plate beyond the beam, (Fig. 3).

EXAMPLE 3.—A wooden beam 10 ins. wide supports a uniform load of 24,000 lbs. What size bearing-plate of cast iron should be used on common brickwork?

Ans. Load on bearing plate $=24{,}000\div 2=12{,}000$ lbs.
Area of plate $=12{,}000\div 100=120$ sq. ins.
Size of plate, $8'' \times 15''$. $P=\dfrac{15-10}{2}=2\tfrac{1}{2}''$.

$$t=.024\times 2\tfrac{1}{2}\sqrt{100}=.024\times 2\tfrac{1}{2}\times 10=.6 \text{ ins.}$$

When the theoretical thickness is less than an inch, the plate had better be made 1 inch thick.

EXAMPLE 4.—A 24-inch 80-lb. steel beam supports a distributed load of 60,000 lbs. What size of bearing-plate should be used on brickwork capable of sustaining 150 lbs. per square inch?

Ans. Load on plate $=60{,}000\div 2=30{,}000$ lbs.
Area of plate $=30{,}000\div 150=200$.
Make size of plate 12×17 ins. Width of beam flange is 7 ins.

$$\text{Hence } P=\frac{17-7}{2}=5 \text{ ins.}$$

For cast iron $t=.024\times 5\sqrt{150}=1.47$ ins.

For steel $t=.0137\times 5\sqrt{150}=0.84$ ins. or $\tfrac{7}{8}$ inch.

The load on the plate equals the end reaction of beam, which is one-half of a distributed or centre load. When the load is irregularly applied the reaction may be computed as explained under supporting forces, Chapter IX.

The following table gives the standard sizes for steel bearing-plates under I-beams and channels as recommended by the Carnegie Steel Co. and Jones & Laughlins, and the bearing values for three grades of masonry. When the reaction of the beam exceeds the safe bearing value given in the table, the size and thickness of the plate should be determined by the foregoing rule. If the reaction is less than the bearing value, the size of the plate can be reduced.

As the reactions vary with the span of the beam, such a table as the following should be used with caution, and the reaction always compared with the bearing value of the plate:

TABLE II.—STANDARD STEEL WALL BEARING-PLATES.

Depth of beam or channel.	Bearing on wall.	Plates.		Safe bearing values in tons for plates resting on			Weights, pounds.
		Size.	Thickness.	Common brickwork. 100 lbs.	First-class brickwork. 150 lbs.	Ordinary masonry. 250 lbs.	
3″, 4″, 5″, and 6″	6″	6″ × 6″	* ⅜″	1.8	2.7	4.5	4
3″, 4″, 5″, and 6″	6″	6″ × 6″	½″	1.8	2.7	4.5	5
7″ and 8″	8″	8″ × 8″	½″	3.2	4.8	8.0	9
7″ and 8″	8″	8″ × 8″	¾″	3.2	4.8	8.0	14
9″ and 10″	8″	8″ × 12″	½″	4.8	7.2	12.0	14
9″ and 10″	8″	8″ × 12″	¾″	4.8	7.2	12.0	20
12″ I 31.5 lbs.	12″	12″ × 12″	½″	7.2	10.8	18.0	20
12″ I 31.5 lbs.	12″	12″ × 12″	¾″	7.2	10.8	18.0	31
12″ I 40 lbs., 15″ I 42 lbs.	12″	12″ × 16″	¾″	9.6	14.4	24.0	41
12″ I 40 lbs., 15″ I 42 lbs.	12″	12″ × 16″	1 ″	9.6	14.4	24.0	54
15″ I 60 and 80 lbs	12″	12″ × 18″	¾″	10.8	16.2	27.0	46
15″ I 60 and 80 lbs	12″	12″ × 18″	1 ″	10.8	16.2	27.0	61
18″, 20″, and 24″	16″	16″ × 16″	1 ″	12.8	19.2	32.0	73

* Use the thicker plate for bearing values exceeding those given under common brickwork.

Bearing-plates on brickwork may be considerably reduced in size by placing a strong flat stone under them. The area of the stone should be proportioned by the above rule, and the thickness of the stone should be at least equal to its projection beyond the iron plate.

BEDDING.

Base-plates should be bedded or grouted in cement from one-half to three-quarters of an inch thick, the plate to be rammed down solid, true and level. Web-plates should have holes in the bottom, as shown in Fig. 1, to show if the cement is distributed evenly under the plate.

Bearing-Brackets on Cast-iron Columns.

Fig. 4 shows the usual method of connecting iron floor-beams and girders with cast-iron columns. The ends of the beam and girder rest on plates P cast on the columns, and the plates are supported by cast brackets C, so that no transverse strain can come upon the plate. For single beams one bracket is sufficient; for double beams, or for wide beams or riveted girders, two brackets should be used. The ends of the beams and girders are fastened

to the column by bolting to the lugs *L*, which are also cast on the column. (See also Fig. 8, Chapter XIV.)

As the plates can resist but little transverse strain, it is evident that the strength of the support consists in the resistance of the brackets and plate to being sheared or sliding down on the column, and also on the resistance of the bracket to crushing. The

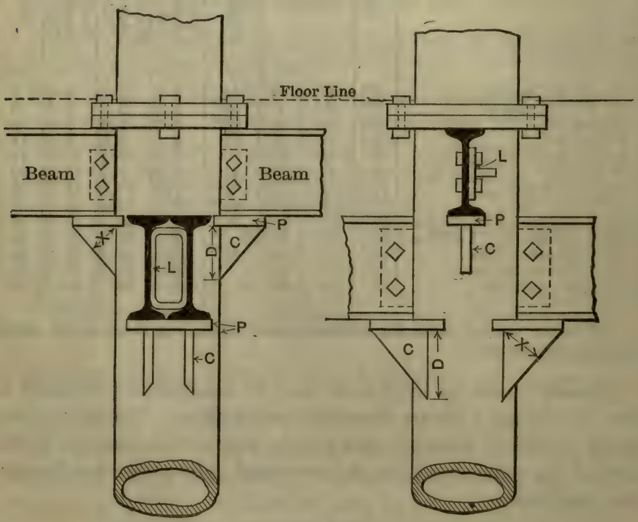


Fig. 4.

thickness of the plate and brackets should not be less than the thickness of the body of the column, and this simple rule will generally insure sufficient strength for supporting the beams or girders. In case of very heavily loaded beams or girders, it would be well, however, to calculate the resistance of the support both to shearing and crushing.

Both the plate and bracket would offer resistance to shearing, but the author advocates considering only the resistance of the bracket. The resistance of a single bracket to shearing is equal to the height *D* multiplied by the thickness of the plate, and the product by 7,000 pounds. Thus if the length *D* is six inches (which should be about the minimum length), and the thickness of the bracket one inch, the shearing area would be six inches,

which, multiplied by 7,000, gives 42,000 pounds as the safe strength of one bracket.

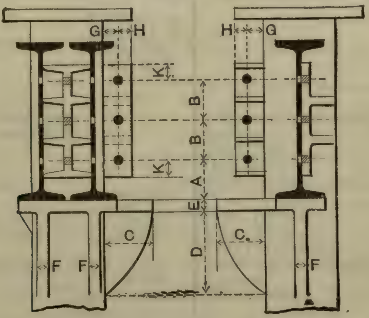
The resistance to crushing may be found by multiplying the distance X by the thickness of the bracket and the product by 13,000. Thus, if X is four inches and the thickness one inch, the resistance to crushing would be 52,000 pounds. Such a bracket would support the end of a 20-inch light steel beam of 16 feet span under its full load; for heavier beams the thickness of the bracket and also the length D should be increased.

Bevel of Brackets.—If the plate P, on which the beam rests, is cast square to the column, then, when the beam deflects, the load will be brought on the extreme outer edge of the column. To avoid this the shelf should be sloped downward, away from the column, with a bevel of $\frac{1}{8}$-inch per foot.

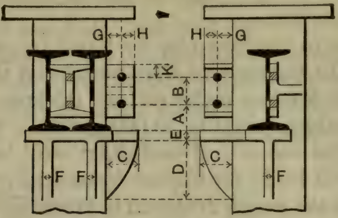
TABLE III.—STANDARD CONNECTIONS TO CAST-IRON COLUMNS.

The following table, published by the Passaic Rolling Mill Co., will be found useful, when detailing cast-iron columns.

ALL DIMENSIONS ARE IN INCHES.



Depth of beam.	A	B	C	D	E	F	G	H	K	Thickness of lugs.	Holes cored for ¾″ bolts.
20	5	5	6	10½	1½	1½	2	1½	2	1	
18	4	5	6	10½	1½	1½	2	1½	2	1	
15	4	3½	5½	9½	1½	1¼	2	1½	1¾	1	
12	3	3	4½	7¾	1¼	1¼	2	1½	1½	1	



Depth of beam.	A	B	C	D	E	F	G	H	K	Thickness of lugs.	
10	$3\frac{1}{4}$	$3\frac{1}{2}$	4	7	$1\frac{1}{4}$	1	2	$1\frac{1}{2}$	$1\frac{1}{2}$	1	Holes cored for $\frac{3}{4}''$ bolts.
9	3	3	4	7	1	1	2	$1\frac{1}{2}$	$1\frac{1}{2}$	1	
8	$2\frac{1}{2}$	3	4	7	1	1	2	$1\frac{1}{2}$	$1\frac{1}{2}$	$\frac{3}{4}$	
7	$2\frac{1}{4}$	$2\frac{1}{2}$	4	7	1	1	2	$1\frac{1}{2}$	$1\frac{1}{4}$	$\frac{3}{4}$	

CHAPTER XIV.

STRENGTH OF POSTS, STRUTS, AND COLUMNS.

DETAILS OF CONNECTIONS AND BASE-PLATES.

As the strength of a post, strut, or column, depends primarily upon the resistance of the given material to crushing, we must first determine the ultimate crushing strength of all materials used for this purpose.

The following table gives the strength for all materials used in building, excepting brick, stone, and masonry, which will be found in Chapter V.

TABLE I.—AVERAGE ULTIMATE CRUSHING LOADS, IN POUNDS PER SQUARE INCH, FOR BUILDING-MATERIALS.

Material.	Crushing weight, in lbs. per sq. inch.	Material.	Crushing weight, in lbs. per sq. inch.
	C.		*C.*
For Stone, Brick, and Masonry see Chap. V.		Woods (*continued*).	
		Cypress	3,375
		Hemlock	3,000
		Oak, white	4,000
Metals.		Pine, Georgia yellow	5,000
Cast iron	80,000	Pine, Oregon	4,500
Wrought iron	36,000	Pine, Norway	3,800
Steel (rolled shapes)	48,000	Pine, white	3,500
		Pine (Colorado)	3,150
Woods (Endways).*		Redwood (California)	3,000
Cedar	3,500	Spruce	4,000
Chestnut	4,000	Whitewood	3,000

* For crushing resistance of timbers perpendicular to the grain see Table VI.

The values for cast iron, wrought iron, and steel are those generally used, although a great deal of iron is stronger than this. The values for woods are those recommended by leading engineers, and may be considered as a fair average of the results obtained by experiment on full-size pieces of merchantable lumber. The values for yellow pine, white pine, white oak, and cypress were

obtained from results of the tests conducted since 1890 by the U. S. Forestry Division. The values for the other woods were compiled from the best test data available, and are believed to be as near the actual strength of ordinary full-size timbers as can be determined.

The values for wood are for dry timber. *Wet timber* is only about one-half as strong to resist compression as dry timber, and this fact should be taken into account when using green timber.

The strength of a column, post, or strut, depends, in a large measure, upon the proportion of the length to the diameter of least thickness. Up to a certain length, failure occurs simply by compression, and above that length by first bending and then breaking.

Wooden Columns.

For wooden columns, where the *length is not more than twelve times the least thickness,* the strength of the column or strut may be computed by the rule,

$$\text{Safe load} = \frac{\text{area of cross-section} \times C}{\text{factor of safety}} \qquad (1)$$

where C denotes the strength of the given material as given in Table I.

The factor of safety to be used depends upon the place where the column or strut is used, the load which comes upon it, the quality of the material, and, in a large measure, upon the value taken for C.

For lumber of ordinary quality, and containing no very bad knots, the author would recommend that a factor of safety of five be used; or, in other words, that the safe stress per square inch of section area be made one-fifth of the values given in Table I.

If the post is badly season-checked, cross-grained, or contains bad knots, a larger factor, say six or seven, should be used. The character of the load should also be taken into consideration in determining the factor of safety. Thus the author would use a larger factor for a post supporting a brick wall than for one supporting a floor, as in the former case the *full load* is at all times on the post, and the least reduction of its sectional area in case of fire might cause it to give way. Columns supporting

machinery, or struts in railway bridges, should have a factor of safety of from 6 to 8, if the values of C, given in Table I., are used.

EXAMPLE 1.—What is the safe load for a hard-pine post 10×10 inches, 12 feet long, using a factor of safety of 5?

Ans.—Area of cross-section = 100 sq. ins.; safe load per sq. in. $= \frac{5000}{5} = 1000$; $1000 \times 100 = 100{,}000$ lbs.

EXAMPLE 2.—It is required to support a brick wall weighing 80,000 lbs. by an Oregon pine post 11 feet long. What should be the size of the post?

Ans.—We would recommend a factor of safety of 6. Then safe resistance per sq. in. of section area $= \frac{4500}{6} = 750$; $\frac{80{,}000}{750} = 106$ sq. ins. required in section of post, or say a 10×11 or 9×12 post.

Strength of Wooden Posts over Twelve Diameters in Length.

When the *length* of a post *exceeds twelve times its least thickness* or diameter, the post is liable to bend under the load, and hence to break under a less load than would a shorter column of the same cross-section.

To deduce a formula which would make the proper allowance for the length of a column has been the aim of many engineers, but their formulæ have not been verified by actual results.

Until within two or three years the formulæ of Mr. Lewis Gordon and Mr. C. Shaler Smith have been generally used by engineers, but the extensive series of tests made on the Government testing machine at Watertown, Mass., on full-sized columns, show that these formulæ do not agree with the results there obtained.

Mr. James H. Stanwood, Instructor in Civil Engineering, Mass. Institute of Technology, in the year 1891 platted the values of all the tests made at the Watertown Arsenal up to that time on full-size posts. From the drawing thus obtained he deduced the following formula for *yellow pine* posts:

$$\text{Safe load per square inch} = 1000 - 10 \times \frac{\text{length in inches}}{\text{breadth in inches}}. \qquad (2)$$

The author has carefully compared this formula with the results of actual tests, and with other formulæ, and believes that it meets the actual conditions more nearly than any other formula, and

he has therefore discarded the tables of wooden posts given in the earlier editions of this work and prepared the following tables for the strength of round and square posts of sizes coming within the range of actual practice.

For other sizes the loads can easily be computed by the formula.

The loads for Texas pine, oak and white pine posts were computed by the following formulæ:

For Texas (yellow) pine:

$$\text{Safe load per square inch} = 850 - 8.5 \times \frac{\text{length in ins.}}{\text{breadth in ins.}}. \quad (3)$$

For oak and Norway pine:

$$\text{Safe load per square inch} = 750 - 7.5 \times \frac{\text{length in ins.}}{\text{breadth in ins.}}. \quad (4)$$

For white pine and spruce posts:

$$\text{Safe load per sqnare inch} = 625 - 6 \times \frac{\text{length in ins.}}{\text{breadth in ins.}}, \quad (5)$$

in which the *breadth* is the least side of a rectangular strut, or the diameter of a round post. The round posts were computed for the half-inch, to allow for being turned out of a square post, of the size next larger.

The formulæ were only used for posts exceeding 12 diameters for yellow pine, and ten diameters for other woods.

For posts having bad knots, or other defects, or which are known to be eccentrically loaded, a deduction of from 10 to 25 per cent. should be made from the values given in the tables.

TABLE II.—SAFE LOAD IN POUNDS FOR YELLOW PINE AND OREGON PINE POSTS (ROUND AND SQUARE.)*

Size of Post in inches.	Length of Post in feet.								
	8	10	12	14	15	16	18	20	24
4×6......	18,200	16,800	15,360						
5½ round..	19,590	18,760	17,550	16,500					
6×6.....	30,200	28,800	27,400	25,900	25,200	24,500			
6×8.....	40,300	38,400	36,500	34,600	33,600	32,600			
6×10.....	50,400	48,000	45,600	43,200	42,000	40,800			
7½ round..	38,540	37,130	35,710	34,300	33,590	32,890			
8×8.....	64,000	54,400	52,500	50,600	49,600	48,600	46,700		
8×10.....	80,000	68,000	65,600	63,200	62,000	60,800	53,400		
8×12.....	96,000	81,600	78,700	76,800	74,400	73,000	70,100		
9½ round..	70,900	61,970	60,190	58,350	57,429	56,580	54,800		
10×10....	100,000	100,000	85,600	83,200	82,000	80,800	78,400	76,000	
10×12....	120,000	120,000	102,700	99,800	98,400	97,000	94,100	91,200	
10×14....	140,000	140,000	119,800	116,500	114,800	113,100	109,800	106,400	
11½ round	103,900	103,900	90,912	88,730	87,690	86,550	84,160	82,290	
12×12....	144,000	144,000	144,000	123,800	122,400	121,000	118,100	115,200	109,440
12×14....	168,000	168,000	168,000	144,500	142,800	141,100	137,800	134,400	127,680
12×16....	192,000	192,000	192,000	165,100	163,200	161,300	157,400	153,600	145,920
14×14....	196,000	196,000	196,000	196,000	170,900	169,100	165,800	162,400	155,800
16×16....	256,000	256,000	256,000	256,000	229,100	225,300	221,400	217,600	209,900
18×18....	324,000	324,000	324,000	324,000	324,000	289,400	285,100	280,800	272,160
20×20....	400,000	400,000	400,000	400,000	400,000	400,000	356,800	352,000	342,400

* These two woods appear to be of about equal strength for posts exceeding 12 diameters in height.

TABLE III.—SAFE LOAD FOR TEXAS (YELLOW) PINE POSTS (ROUND AND SQUARE).

Size of post in inches.	Length of post in feet.								
	8	10	12	14	15	16	18	20	24
4×6.....	15,500	14,280	13,050						
5½ round..	16,650	15,790	14,900	14,030					
6×6.....	25,704	24,480	23,256	22,032	21,420	20,808			
6×8.....	34,272	32,640	31,008	29,376	28,560	27,744			
6×10.....	42,840	40,800	37,760	36,720	35,700	34,680			
7½ round..	32,740	31,540	30,340	29,140	28,540	27,940	26,740		
8×8.....	47,870	46,240	44,600	42,970	42,160	41,340	39,710		
8×10.....	59,840	57,800	55,760	53,720	52,700	51,680	49,640		
8×12.....	71,808	69,360	66,910	64,460	63,240	62,000	59,560		
9½ round..	54.150	52,650	51,150	49,580	48,820	48,070	46,570		
10×10....	85,000	78,800	72,760	70,720	69,700	68,680	66,640	64,600	
10×12....	102,000	89,760	87,300	84,860	83,640	82,400	80,000	77,500	
10×14....	119,000	104,700	101,860	99,000	97,580	96,150	93,300	90,400	
11½ round	88,290	79,100	77,250	75,400	74,470	73,550	71,700	69,850	66,160
12×12....	122,400	110,160	107,700	105,260	104,040	102,800	100,360	97,920	93,000
12×14....	142,800	128,520	125,660	122,800	121,380	119,950	117,100	114,240	108,520
12×16....	163,200	146,880	143,600	140,350	138,720	137,080	133,800	130,560	124,030
14×14....	166,600	166,600	149,450	146,600	145,180	143,760	140,900	138,080	132,400
14×16....	190,400	190,400	170,800	167,500	165,900	164,300	161,000	157,800	151,300
16×16....	217,600	217,600	217,600	194,700	193,000	191,400	188,200	184,900	178,400

TABLE IV.—SAFE LOAD IN POUNDS FOR OAK AND NORWAY PINE POSTS (ROUND AND SQUARE).

Size of post in inches.	Length of post in feet.								
	8	10	12	14	15	16	18	20	24
4×6.	13,680	12,600	11,520						
5½ round. .	14,700	13,900	13,160	12,370					
6×6.	22,680	21,600	20,520	19,440	18,900	18,360			
6×8.	30,240	28,800	27,360	25,920	25,200	24,480			
6×10.	37,800	36,000	34,200	32,400	31,500	30,600			
7½ round. .	28,900	27,850	26,780	25,720	25,190	24,660			
8×8.	42,240	40,768	39,360	37,880	37,120	36,480	35,000		
8×10.	52,800	50,960	49,200	47,360	46,400	44,600	43,760		
8×12.	63,360	61,152	59,040	56,830	55,680	54,720	52,500		
9½ round. .	47,960	46,440	45,160	43,740	43,100	42,400	41,120		
10×10. . . .	75,000	66,000	64,200	62,400	61,500	60,600	58,800	57,000	
10×12. . . .	90,000	79,200	77,040	74,880	73,800	72,720	70,560	68,400	
10×14. . . .	105,000	92,400	89,880	87,360	86,100	84,840	82,320	79,800	
11½ round.	77,925	69,820	68,160	66,490	65,770	64,833	63,170	61,600	
12×12. . . .	108,000	108,000	95,040	92,880	91,700	90,700	88,560	86,400	82,080
12×14. . . .	126,000	126,000	110,800	108,300	107,000	105,840	103,300	100,802	95,760
12×16. . . .	144,000	144,000	126,700	123,800	122,300	120,900	118,000	115,200	109,400
14×14. . . .	147,000	147,000	147,000	129,300	128,100	127,000	124,400	121,900	116,800
16×16. . . .	192,000	192,000	192,000	192,000	170,500	168,900	166,100	163,000	157,400
18×18. . . .	243,000	243,000	243,000	243,000	243,000	217,000	213,800	210,600	204,100
20×20. . . .	300,000	300,000	300,000	300,000	300,000	300,000	267,600	264,000	256,000

TABLE V.—SAFE LOAD IN POUNDS FOR WHITE PINE AND SPRUCE POSTS (ROUND AND SQUARE).

Size of post in inches.	Length of post in feet.								
	8	10	12	14	15	16	18	20	24
4×6.	11,520	10,550	9,800	8,700					
5½ round. .	12,350	11,730	11,180	10,490					
6×6.	19,080	18,216	17,352	16,490	16,050	15,620			
6×8.	25,440	24,290	23,140	21,980	21,400	20,830			
6×10.	31,800	30,360	28,920	27,480	26,760	26,040			
7½ round. .	24,220	23,380	22,540	21,660	21,260	20,820			
8×8.	35,450	34,300	33,150	32,000	31,420	30,850	29,700		
8×10.	44,320	42,480	41,440	40,000	39,280	38,560	37,120		
8×12.	53,180	51,450	49,730	48,000	47,140	46,270	44,544		
9½ round. .	40,000	39,000	37,860	36,800	36,230	35,730	34,670		
10×10. . . .	62,500	55,400	53,960	52,520	51,800	51,080	49,640	48,200	
10×12. . . .	75,000	66,480	64,800	63,000	62,160	61,300	59,570	57,840	
10×14. . . .	87,500	77,560	75,600	73,500	72,520	71,510	69,500	67,480	
11½ round.	64,930	58,390	57,140	55,800	55,170	54,550	53,100	51,950	
12×12. . . .	90,000	90,000	79,780	78,000	77,180	76,320	74,590	72,860	69,400
12×14. . . .	105,000	105,000	93,170	91,050	90,050	89,000	87,020	85,000	80,900
12×16. . . .	120 000	120,000	106,300	104,000	102,900	101,700	99,400	97,150	92,500
14×14. . . .	122,500	122,500	110,350	108,350	107,400	106,400	104,460	102,300	98,400
16×16. . . .	160,000	160,000	160,000	143,870	142,590	141,570	139,260	136,960	132,360
18×18. . . .	202,500	202,500	202,500	202,500	183,060	181,760	179,170	176,580	171,400
20×20. . . .	250,000	250,000	250,000	250,000	250,000	250,000	224,500	221,200	215,200

Eccentric Loading.

When the load on a post is applied in such a way that it is not distributed uniformly over the end of the post the loading is called eccentric, and the effect on the post is much more injurious than if the load were uniformly distributed. When a post supports a girder on one side only, or when the weight from one girder is much more than from the other, the load becomes eccentric, and the sectional area of the post should be increased, to resist the bending stress due to the eccentricity.

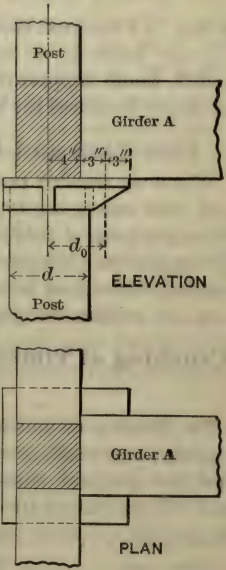


Fig. 1

When the eccentric load is applied as in Fig. 1, the sectional area of a square or rectangular post should be computed by the following formula:

Sectional area of post in square inches

$$=\frac{W}{p}+\frac{6W_1\times d_0}{p\times d}, \qquad (6)$$

in which W = total load on post in lbs.;
W_1 = eccentric load in lbs.;
p = safe stress in lbs. per sq. inch.;
d_0 = distance from centre of post to centre of bearing in ins.;
d = side of post parallel with girder.

In assuming the value of p, the probable ratio of the side of the post to the length should be taken into account. Thus if it is probable that the length will not exceed twelve times the side (both being measured in inches) for yellow pine or Oregon pine posts, or 10 diameters for other woods, then the value of p for short posts may be taken. If the ratio will probably be greater than this, then the probable ratio should be roughly calculated and p computed for that ratio by the formula given for posts more than 10 diameters in length.

Example 3.—The post P_1, Fig. 1, supports a total load on its cap-plate of 60,000 lbs., including the reaction from girder A of 12,000 lbs. What should be the size of the post for Oregon pine,

and length of 12 ft.? *Ans.*—As it is probable that the post will have to be at least 10 ins. square, we will assume 1,000 lbs. for p, and 10 ins. for d. W_1 will = 12,000 lbs. and d_0 7 ins.

$$\text{Then sectional area} = \frac{60,000}{1,000} + \frac{6 \times 12,000 \times 7}{1,000 \times 10} = 60 + 50.4 = 110.4$$

sq. ins. Or the post should be 11×11 or 10×12 ins.

From Table I. we see that an 8×10 post, concentrically loaded, would support 65,000 lbs., hence the eccentric load from the girder increases the size of the post from 8×10 to 10×12 ins.

Iron Caps and Bolsters for Wooden Posts.

Whenever wooden posts are used in tiers, one above another, each post except the top one should have an iron cap-plate, and the upper posts should set on the cap of the post below, and *not* on the girder. Where a wooden post supports only a girder a wooden bolster may be used in place of the cap. Details of post caps and bolsters are shown in Chapter XXII.

Crushing of Timber Perpendicular to the Grain.

TABLE VI.

The bearing of wooden girders, the ends of posts resting on a girder, and washers on truss rods, should be proportioned so that the quotient obtained by dividing the load by the bearing area will not exceed the following safe unit strains:

White oak..........	600 lbs.	Colorado pine.......	200 lbs.
Yellow pine.........	500 "	Spruce.............	250 "
Oregon pine........	400 "	Hemlock...........	200 "
Norway pine.......	250 "	Cypress............	200 "
White pine.........	200 "	Redwood...........	175 "

Cast-iron Columns and Posts.

Advantages and Disadvantages.—Although steel is being more largely used every year for the upright supports in buildings, it will probably never entirely supplant the cast-iron post, and, in fact, it is still a disputed question whether a steel post is better than one of cast iron for buildings of moderate height.

For skeleton construction, when the height of the building exceeds twice its width, it seems unquestionable that the riveted steel column, "breaking joint" in alternate stories, and with riveted connections with the beams and girders, is much the best; but for the larger proportion of the buildings in which iron posts

are used cast iron possesses advantages which the author believes are not exceeded by the riveted steel post.

The most important of these advantages are—low cost, quickness of production, adaptability to any desired shape, and ease in making connections.

Cast-iron columns when unprotected will also resist the action of fire better than unprotected steel columns, as has been quite conclusively demonstrated by the experiments of Prof. Bauschinger, of Munich.* Cast iron three quarters of an inch or more in thickness is also practically uninjured by rust, while it is claimed that wrought iron or steel may be almost destroyed by it, unless kept constantly protected by some coating impervious to moisture.

For unprotected columns cast iron may be made in more attractive shape than steel columns, without additional cost.

The disadvantages of cast-iron columns, as found in practice, are lack of uniformity in the metal, the danger of shrinkage strains, and the difficulty of making rigid connections.

There are many contingencies which may arise in the manufacture of cast-iron columns which preclude perfect uniformity in the product.

Among these are unevenness in the thickness of the metal, which has sometimes been found to be very different on one side of a round column from that on the opposite side. The presence of confined air, producing "blow-holes" and "honeycomb," and the collection of impurities at the bottom of the mould, are also frequent sources of weakness in cast iron.

By careful inspection and by boring the columns these defects can be discovered and imperfect columns rejected.

The most critical condition is that due to the unequal contraction of the metal during the process of cooling, thereby giving rise to initial strains, at times of sufficient force to produce rupture in the column or in its lugs on the slightest provocation.

In many cases the trouble is due to faulty designing or careless-

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* Architect C. A. Ziegler, of Philadelphia, in commenting on the conflagration at Paterson, N, J., in Feb., 1902, says, in the *Brickbuilder* for March of that year: "The manner in which cast-iron columns and girders withstood the flames is most marvelous. The great majority that I saw were not only intact but were as good as new, although some had apparently fallen from great heights.

"The steel work, on the contrary [in modern fireproof buildings], wherever it was reached by the heat, buckled and fell, heavy steel girders and posts being twisted and knotted like whipcords."

ness in the execution of the work; yet, even under favorable conditions, it is so difficult to secure equal radiation from the moulds in all directions that castings entirely exempt from inherent shrinkage strains are probably seldom produced.

The most serious difficulties met with in using cast columns in tall buildings are the difficulty of making true and rigid end connections and the unreliability of the brackets which support the beams and girders. By skilful designing and careful workmanship these difficulties may to a considerable extent be overcome, but cast columns can never in these respects be made to equal the best forms of steel columns.

The length of cast-iron columns in inches should not exceed thirty-six times their diameter, or least dimension.

Shapes of Cast Columns.—Cast-iron columns for buildings have been made in all of the nine shapes shown in Fig. 2, although solid columns are seldom made, and only for very small diameters.

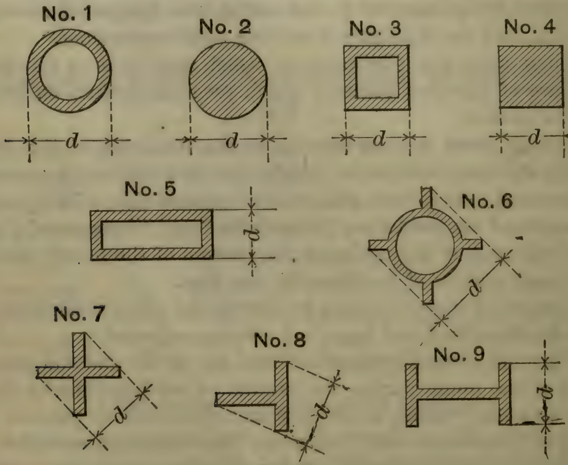

Fig. 2

For interior unprotected columns the hollow cylindrical shape probably meets the usual requirements better than any other.

For exterior columns, as in store fronts, the rectangular shape, No. 5, is more generally used, in order to give a good bearing for the beams supporting the wall above.

For fire-proof buildings in which cast-iron columns are used the author believes that the H-shaped No. 9 is the most desirable, because of the following advantages:

1. Being entirely open, with both the interior and exterior surfaces exposed, any inequalities in thickness can be readily discovered, and the thickness itself easily measured, thus obviating any necessity for boring, and rendering the inspection of the columns much less tedious.

2. The entire surface of the column can be protected by paint.

3. When built in brick walls the masonry fills all voids, so that no open space is left, and if the column is placed as shown in Fig. 3, only the edge of the column comes near the face of the wall.

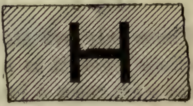

Fig. 3

4. Lugs and brackets can be cast on such columns better than on circular columns, especially for wide and heavy girders.

5. The end connections of the columns do not require projecting rings, or flanges, which are often objectionable in circular columns.

The cost of columns of this shape should not exceed that of circular columns of the same strength.

The column may be fire-proofed in the same way as the Z-bar column, which it much resembles. The space occupied by the column slightly exceeds that of both the cylindrical and the Z-bar column, but not enough to be of any serious consequence.

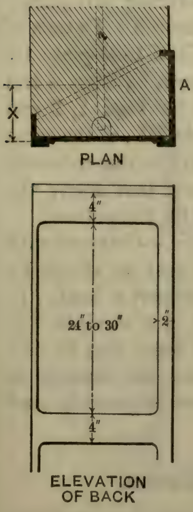

Fig. 4

Pilasters.—Pilasters, or columns without a back, are often used as a facing to the brick walls at the sides of store fronts. If such pilasters are used to support a girder, the inner side, A, Fig. 4, should be wide enough to receive the girder, and the back should be stiffened by cast ribs about every thirty inches in height. If the load on the girder is very great it will be much better construction to build an H-shaped column in the wall, to support the girder, and put up a false front for appearance, as shown in section by Fig. 5. To compute the strength of a pilaster of the section shown by Fig. 4, divide the length in inches by the distance X, and from Table VII. find the ultimate strength per square inch of metal for that ratio. Then multiply the sectional area of the front and two sides by this

value, and divide by 10 or 12 for a factor of safety. The pilaster should be anchored to the brick wall by long iron anchors hooked through a lug cast on the inside of the face. The interior of the pilaster should also be filled with brickwork.

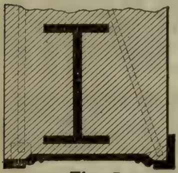

Fig. 5

Connections of Cast-iron Columns.—The bearing of cast-iron columns should always be turned true to the axis of the column. Where only two stories of columns are used, and the joint is at a floor level, it is not necessary to bolt the columns together. For such cases a joint made as shown by Fig. 6 will generally be found satisfactory. If more than the two stories of columns are used, or the column is not well braced where the joint occurs, the columns should be bolted together by four ¾-inch bolts for columns 10 inches in diameter or less, and six bolts for 12-inch and larger columns. A desirable section for such joints is that shown by Fig. 7. By keeping the lugs a quarter of an inch from the end, less facing is required, and a better bearing is ensured. Details of end connections, brackets and base-plate for H-shaped columns are shown by Figs. 8 and 9, and for round columns by Figs. 1, 2 and 4, Chap. XIII., also by Table III, of the same chapter. For convenience in erecting columns, the joint is generally placed just above the beams or girder supported by the column.

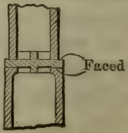

Fig. 6

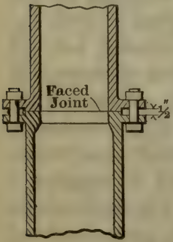

Fig. 7

Projecting Caps and Bases.—Columns with ornamental caps and bases, should *never* be cast as shown by the section Fig. 10, i. e., if the column is to support a load. In all bearing columns, the core should extend in a straight line from end to end. Plain moulded caps and bases may be cast solid as in Fig. 11; if more ornamental caps are desired, or heavy projecting bases, they should be cast separately and attached to the straight column by screws.

Strength of Cast-Iron Columns.

The ultimate resistance of cast.iron to crushing is generally taken at 80,000 lbs. per square inch, and for posts, pintels, etc.,

where the length is not more than six times the diameter, or breadth, it will usually be safe to figure the working strength at

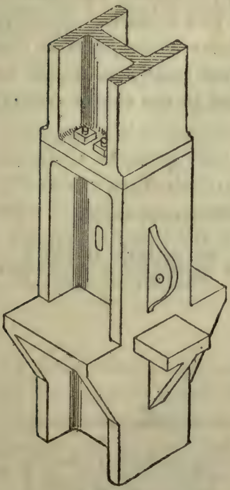

Fig. 8

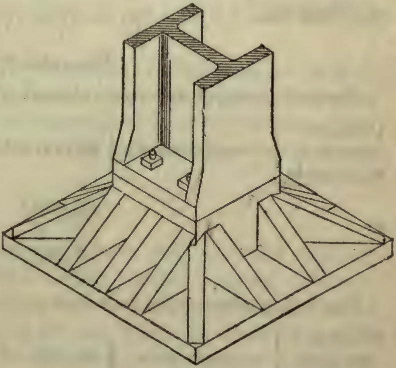

Fig. 9

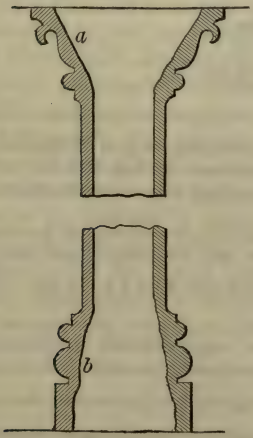

Fig. 10

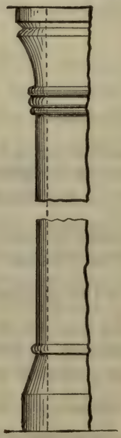

Fig. 11

six tons per square inch of metal. For longer posts, or columns, the strength is affected by the ratio of length to diameter, but to

just what extent is not definitely known, hence all formulas for columns must be more or less theoretical. The consequence is that a great many formulas have been published, and there is none that is universally accepted. The two following, however, are now more commonly adopted than any others, and as they appear to agree as well as any with actual tests, the author has adopted them in place of those presented in the earlier editions of this work.

*Formulas.**

For hollow round cast-iron columns with square ends,

$$\left.\begin{matrix}\text{Ultimate}\\ \text{strength}\\ \text{in pounds}\end{matrix}\right\} = \text{metal area} \times \left[80{,}000 \div 1 + \frac{\text{sq. of lgth. in ins.}}{800 \times \text{sq. of diam.}}\right], \quad (7)$$

or

$$\frac{80{,}000A}{1 + \dfrac{l^2}{800d^2}}.$$

For hollow rectangular cast-iron columns with square ends,

$$\left.\begin{matrix}\text{Ultimate}\\ \text{strength}\\ \text{in pounds}\end{matrix}\right\} = \text{metal area} \times \left[80{,}000 \div 1 + \frac{\text{sq. of lgth. in ins.}}{\begin{matrix}1{,}067 \times \text{sq. of least}\\ \text{side in inches}\end{matrix}}\right], \quad (8)$$

or

$$\frac{80{,}000A}{1 + \dfrac{l^2}{1{,}067d^2}}.$$

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* The tables in the handbooks of the Cambria Iron Co., the Carnegie Steel Co., Jones & Laughlins, and the Passaic Rolling Mill Co. are based on formulas (7) and (8), and they have been adopted in the Boston building laws.

The values obtained by these formulas will be slightly in excess of those given in the Chicago building law, and considerably less than those permitted by the building law of Greater New York $\left(p = 11{,}300 - \frac{l}{r}\right)$.

In 1898 Prof. W. H. Burr made an analysis of the results of a number of experiments on full-size, hollow, round cast-iron columns made at the Watertown Arsenal and Phœnixville, and by plotting the results found that a straight-line formula having the equation $p = 30{,}500 - 160\frac{l}{d}$ represents the average of the plotted results. With a factor of safety of four this would become $p = 7{,}625 - 40\frac{l}{d}$.

According to Prof. Burr's analysis the values for p given in the fourth

For solid cylindrical cast-iron columns,

$$\left.\begin{array}{c}\text{Ultimate}\\ \text{strength}\\ \text{in pounds}\end{array}\right\} = \text{metal area} \times \left[80{,}000 \div 1 + \frac{\text{sq. of lgth. in ins.}}{266 \times \text{sq. of diam.}} \right]. \quad (9)$$

For the star, T and H shape, use formula (7), taking d as shown in Fig. 2 for the diameter.

The safe load is generally taken at *one eighth* of the ultimate strength or breaking load.

Eccentric Loading.—Cast-iron columns should not be loaded with heavy eccentric loads, i.e., a load applied on one side of the column without a corresponding load on the other side, as cast iron is unable to resist very great bending strains.

Tables.

As the allowable pressure *per square inch of metal* depends upon the ratio of length to diameter, without regard to actual dimensions, i.e., it would be the same for a column 6 ins. in diameter and 12 feet long, as for one 8 ins. in diameter and 16 feet long, it is practicable to prepare a table which will give the value of the portions of formulas (7) and (8) inclosed in brackets for all ratios of diameter to length, which will very much simplify the computation for any particular column.

Table VII. has been computed by means of the formulas for ratios of length to diameter varying from 8 to 36, and the same result will be obtained by using the values given in this table as by using the corresponding formula.

To use this table it is only necessary to divide the length of the column in inches by the least thickness or diameter also in inches, and opposite the number in column 1 coming nearest to the quotient find the safe strength per square inch for the column. Multiply this load by the metal area in the cross-section of the column, and the result will be the safe load for the column.

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column of Table VII. represent a factor of safety of a little over four for $\frac{l}{d}=20$, and nearly seven for $\frac{l}{d}=36$.

A series of tests on full-size cast-iron columns and brackets was made under the direction of Mr. Stevenson Constable, in December, 1897, a report of which with illustrations may be found in the *Engineering Record* for January 8 and 22, 1898.

EXAMPLE 4.—What is the safe load for a 10-inch cylindrical cast-iron column 15 feet long, the shell being one inch thick?

Ans. The length of the column divided by the diameter, both in inches, is 18, and opposite 18 in Table VII. we find the safe load per square inch for a round column to be 7,117 pounds. The metal area of the column we find to be 28.27 inches; and multiplying these two numbers together, we have for the safe load of the column 201,197 pounds, or about 100.5 tons.

To still further facilitate computations, Tables VIII., IX., and X. have been prepared, which give at a glance the safe loads (based on a factor of safety of 8) for columns of the more common size and length. For lengths between those given in the tables sufficiently accurate results may be obtained by interpolation. For any other factor of safety multiply the safe load given in the table by 8, and divide by the new factor of safety.

TABLE VII.—STRENGTH PER SQUARE INCH OF HOLLOW, ROUND, AND RECTANGULAR CAST-IRON COLUMNS.

(Calculated by Formulas (7) and (8).)

Length in inches divided by external breadth or diameter.	Breaking weight in pounds per square inch.		Safe load in pounds per square inch. Safety factor 8.	
	Round.	Rectangular.	Round.	Rectangular.
8	74,074	75,470	9,259	9,433
9	72,661	74,350	9,082	9,293
10	71,110	73,126	8,888	9,140
11	69,505	71,870	8,688	8,983
12	67,800	70,487	8,475	8,811
13	66,060	69,084	8,257	8,635
14	64,257	67,567	8,032	8,446
15	62,450	66,060	7,806	8,257
16	60,606	64,516	7,576	8,064
17	58,780	62,942	7,347	7,867
18	56,940	61,360	7,117	7,670
19	55,134	59,745	6,892	7,468
20	53,333	58,180	6,666	7,272
21	51,580	56,610	6,447	7,076
22	49,843	55,020	6,230	6,877
23	48,163	53,470	6,020	6,684
24	46,512	51,950	5,814	6,494
25	44,918	50,440	5,614	6,305
26	43,360	48,960	5,420	6,120
27	41,862	47,530	5,233	5,940
28	40,404	46,110	5,050	5,764
29	39,000	44,742	4,875	5,592
30	37,647	43,390	4,706	5,424
31	36,347	42,080	4,543	5,260
32	35,090	40,816	4,386	5,102
33	33,884	39,580	4,235	4,947
34	32,720	38,380	4,090	4,797
35	31,608	37,244	3,951	4,655
36	30,534	36,120	3,817	4,515

TABLE VIII.—SAFE LOAD IN TONS OF 2,000 POUNDS FOR HOLLOW ROUND CAST-IRON COLUMNS WITH SQUARE ENDS.

(Based on formula (7). Safety factor 8.)

Diameter in inches.	Thickness in inches.	Length of column in feet.										Area of metal in inches.	Weight per foot of length.
		6	8	10	12	14	16	18	20	22	24		
5	¾	39	34	29	24	....	....	....	....	....	....	10.0	31.3
	⅞	45	38	32	27	....	....	....	....	....	....	11.3	35.3
5½	¾	46	40	35	30	26	....	....	....	....	....	11.2	35.0
	⅞	52	46	40	34	29	....	....	....	....	....	12.7	39.7
6	¾	52	47	41	36	31	27	24	....	....	....	12.4	38.7
	⅞	60	53	47	41	36	31	27	....	....	....	14.1	44.0
	1	66	59	52	45	39	34	30	....	....	....	15.7	49.0
7	¾	65	60	54	48	43	38	34	....	....	....	14.7	46.0
	⅞	74	68	62	55	49	43	38	....	....	....	16.8	52.6
	1	83	76	68	61	54	48	43	....	....	....	18.8	58.9
8	¾	78	72	67	61	55	50	45	40	36	33	17.1	53.4
	⅞	89	83	76	70	63	57	51	46	41	37	19.6	61.2
	1	100	93	86	79	71	64	58	52	47	42	22.0	68.7
9	⅞	103	98	91	85	80	71	65	59	54	49	22.3	69.8
	1	117	110	103	95	90	80	73	67	61	55	25.1	78.5
	1⅛	129	122	114	105	99	89	81	74	67	61	27.8	87.0
10	⅞	118	112	106	100	93	86	79	73	67	62	25.1	78.4
	1	133	127	120	112	105	97	89	82	76	69	28.3	88.4
	1⅛	147	141	133	125	116	107	99	91	84	77	31.4	98.0
	1¼	161	154	146	136	127	118	109	100	92	84	34.4	107.4
11	1	149	143	137	129	122	114	106	98	91	85	31.4	98.2
	1⅛	165	159	152	144	135	126	118	109	101	94	34.9	109.1
	1¼	182	175	167	158	148	139	129	120	111	103	38.3	119.7
	1⅜	197	190	181	171	161	151	140	130	121	112	41.6	129.9
12	1⅛	184	178	171	163	154	146	137	128	120	112	38.4	120.1
	1¼	202	195	188	179	170	160	150	141	132	123	42.2	131.9
	1⅜	220	212	204	194	184	174	163	153	143	133	45.9	143.4
	1½	237	229	220	210	199	187	176	165	154	144	49.5	154.6
13	1⅛	202	196	190	182	174	165	156	147	138	130	42.0	131.2
	1¼	222	216	209	200	191	181	172	162	152	143	46.1	144.2
	1⅜	242	235	227	218	208	197	187	176	166	156	50.2	156.9
	1½	261	254	245	235	224	213	201	190	179	168	54.2	169.4
14	1¼	242	236	229	221	212	203	193	183	173	164	50.1	156.5
	1⅜	264	258	250	241	231	221	210	199	189	178	54.5	170.4
	1½	285	278	270	260	250	238	227	215	204	193	58.9	184.1
	1⅝	306	298	289	279	268	256	243	231	219	207	63.2	197.4
15	1⅜	268	280	272	264	254	244	234	223	212	203	58.9	183.9
	1½	309	303	295	285	275	264	252	241	229	219	63.6	203.4
	1⅝	332	325	316	306	295	283	271	259	246	235	68.3	213.4
	1¾	354	346	337	327	315	302	288	276	263	251	72.8	227.6
16	1½	333	327	319	310	300	290	278	267	255	243	68.3	213.5
	1⅝	358	351	343	333	322	311	299	286	273	261	73.4	229.3
	1¾	382	375	366	356	344	332	319	306	292	279	78.3	244.8
	1⅞	455	446	435	423	410	395	380	364	347	332	93.2	291.3

TABLE IX.—SAFE LOAD IN TONS OF 2,000 POUNDS FOR HOLLOW SQUARE AND RECTANGULAR CAST-IRON COLUMNS WITH SQUARE ENDS.

(Based on Formula (8). Safety factor 8.)

Size in inches.	Thickness in inches.	Length of column in feet.								Area of metal in inches.	Weight per foot of length.
		8	10	12	14	16	18	20	24		
4× 6	¾	41	34	28	....	....	....	....	....	12.75	39.8
4× 8	¾	51	42	35	....	....	....	....	....	15.75	49.2
4× 9	¾	56	46	39	....	....	....	....	....	17.25	53.9
4×10	¾	60	50	42	....	....	....	....	....	18.75	58.6
4×12	¾	70	59	49	....	....	....	....	....	21.75	68.0
5× 8	¾	64	55	48	41	....	....	....	....	17.25	53.9
	1	81	71	61	53	....	....	....	....	22.00	68.8
5× 9	¾	69	60	52	45	....	....	....	....	18.75	58.6
	1	89	78	67	58	....	....	....	....	24.00	75.0
5×10	¾	75	65	57	49	....	....	....	....	20.25	63.3
	1	96	84	73	63	....	....	....	....	26.00	81.3
5×12	¾	86	74	65	56	....	....	....	....	23.25	72.7
	1	111	97	84	72	....	....	....	....	30.00	93.8
6× 6	¾	63	57	51	45	40	35	....	....	15.75	49.2
	1	80	72	65	57	51	45	....	....	20.00	62.5
6× 8	¾	75	68	60	54	47	42	....	....	18.75	58.6
	1	96	87	78	69	61	54	....	....	24.00	75.0
6× 9	¾	81	73	65	58	51	45	....	....	20.25	63.3
	1	104	94	84	75	66	58	....	....	26.00	81.3
6×10	¾	87	79	70	62	55	49	....	....	21.75	68.0
	1	112	101	91	80	71	63	....	....	28.00	87.5
6×12	¾	99	90	80	71	63	55	....	....	24.75	77.3
	1	129	116	104	92	81	72	....	....	32.00	100.0
6×15	¾	117	106	95	84	74	66	....	....	29.25	91.4
	1	153	138	123	109	97	85	....	....	38.00	118.8
7× 7	¾	80	73	67	61	55	49	44	....	18.75	58.6
	1	102	94	85	78	70	63	57	....	24.00	75.0
7× 9	¾	92	85	77	70	63	57	51	....	21.75	68.0
	1	119	109	100	91	82	74	66	....	28.00	87.5
7×12	¾	111	102	93	85	77	69	62	....	26.25	82.0
	1	144	133	121	110	99	89	80	....	34.00	106.3
8× 8	¾	95	90	83	77	70	64	59	49	21.75	68.0
	1	124	115	107	99	91	83	76	63	28.00	87.5
	1¼	148	140	129	119	109	100	91	76	33.75	105.5
8×10	¾	109	103	95	87	80	73	67	55	24.75	77.3
	1	141	132	122	113	104	95	86	72	32.00	100.0
	1¼	170	161	148	137	125	115	105	87	38.75	121.1
8×12	¾	122	115	106	98	90	82	75	62	27.75	86.7
	1	158	148	138	127	116	107	97	81	36.00	112.5
	1¼	192	181	167	154	142	130	118	98	43.75	136.7

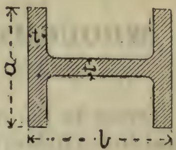
TABLE IX.—SAFE LOAD IN TONS OF 2,000 POUNDS FOR HOLLOW SQUARE AND RECTANGULAR CAST-IRON COLUMNS WITH SQUARE ENDS (*continued*).

(Based on formula (8). Safety factor 8.)

Size in inches.	Thickness in inches.	Length of column in feet.								Area of metal in inches.	Weight per foot of length.
		8	10	12	14	16	18	20	24		
8×16	1	193	181	168	155	142	130	119	99	44.00	137.5
	1¼	236	221	206	190	174	159	145	121	53.75	168.0
9× 9	¾	111	106	99	93	86	80	74	63	24.75	77.3
	1	144	137	129	120	112	103	96	85	32.00	100.0
9×12	1	171	162	153	143	133	123	114	97	38.00	118.8
	1¼	209	198	186	174	162	149	138	118	46.25	144.5
9×16	1	207	196	185	173	161	149	138	117	46.00	143.8
	1¼	254	240	226	212	197	182	168	143	56.25	175.8
10×10	1	165	158	150	142	133	125	117	101	36.00	112.5
	1¼	201	193	183	172	162	152	142	123	43.75	136.7
10×12	1	184	176	167	158	148	139	129	112	40.00	125.0
	1¼	224	214	204	192	181	169	158	137	48.75	152.3
10×15	1	211	202	192	181	170	160	149	129	46.00	143.8
	1¼	258	247	235	222	209	195	182	158	56.25	175.8
10×16	1	220	211	200	189	178	167	155	135	48.00	150.0
	1¼	270	258	245	232	218	204	190	165	58.75	183.6
10×18	1	239	228	217	205	193	181	168	146	52.00	162.5
	1¼	293	280	266	251	236	221	207	179	63.75	199.2
10×20	1	257	246	234	221	208	194	181	157	56.00	175.0
	1¼	316	302	287	271	255	239	223	193	68.75	214.9
10×24	1	294	281	267	252	237	222	207	180	64.00	200.0
	1¼	362	346	329	311	292	274	255	221	78.75	246.1
12×12	⅞	183	177	171	164	156	149	141	126	38.9	121.7
	1	207	201	193	185	177	168	159	142	44.00	137.5
	1¼	253	245	236	223	216	206	195	174	53.75	168.0
	1½	296	288	277	265	253	241	228	204	63.00	196.9
12×15	1	235	228	220	211	201	191	181	162	50.00	156.3
	1¼	288	280	269	258	246	234	222	198	61.25	191.4
12×16	1	245	237	228	219	209	199	188	168	52.00	162.5
12×18	1	263	256	246	236	225	214	203	181	56.00	175.0
12×20	1	282	274	264	253	241	229	217	194	60.00	187.5
12×24	1	320	310	299	287	274	260	246	220	68.00	212.5
14×16	1	268	261	254	246	238	229	219	200	56.00	175.0
14×20	1	307	298	290	281	272	261	250	228	64.00	200.0
14×24	1	345	336	326	316	306	294	280	257	72.00	225.0
16×16	1	300	284	278	271	264	256	247	229	60.00	187.5
16×24	1	380	360	352	344	334	324	313	291	76.00	237.5
18×18	1	340	340	320	314	307	299	291	274	68.00	212.5
20×20	1	380	380	361	356	349	342	334	317	76.00	237.5
20×24	1	420	420	399	393	386	378	369	351	84.00	262.5

TABLE X.—SAFE LOAD IN TONS OF 2,000 POUNDS FOR H-SHAPED CAST-IRON POSTS.

(Based on formula (7). Safety factor 8.)



Size of post in inches. a. b. t.	Area in inches.	Length of post in feet. 10	12	13	14	15	16	18	20
6× 6× ¾	12⅜	41	36	33	31				
1	16	53	46	43	40				
1¼	19⅜	64	56	52	48				
6× 8× ¾	13⅞	46	40	37	34				
1	18	60	52	48	45				
1¼	21⅞	73	63	59	54				
7× 7×1	19	69	62	58	55	52	49	43	38
1¼	23⅛	84	75	71	67	63	59	53	46
7× 9×1	21	76	68	64	61	57	54	48	42
1¼	25⅝	93	83	79	74	70	66	59	51
8× 8× ¾	16⅞	66	60	57	54	51	49	44	39
1	22	86	78	74	70	67	64	57	51
1¼	26⅞	105	95	91	86	82	78	70	63
8×10×1	24	93	85	81	77	73	69	62	56
1¼	29⅜	114	104	99	94	90	85	76	69
1½	34½	134	122	117	111	105	100	89	81
9× 9×1	25	102	94	91	87	83	79	72	66
1¼	30⅝	125	116	111	106	102	97	89	81
1½	36	147	136	130	125	120	114	104	95
9×10×1	26	106	98	94	90	86	83	75	69
1¼	31⅞	130	120	115	111	106	101	92	84
1½	37⅛	153	142	136	130	125	119	108	99
10×10×1	28	118	111	107	103	99	95	88	81
1¼	34¾	145	136	131	127	122	127	108	100
1½	40½	171	160	155	149	144	138	128	117
1¾	46⅜	196	184	177	171	165	158	146	134
10×12×1	30	127	119	115	111	106	102	94	87
1¼	36⅞	156	146	141	136	131	126	116	107
1½	43½	184	172	166	160	154	148	137	126
1¾	49⅞	211	198	191	184	177	170	157	144
2	56	236	222	214	207	199	191	176	162
12×12×1	34	151	144	140	136	132	128	121	113
1¼	41⅞	186	177	172	167	163	158	149	139
1½	49½	220	209	203	198	193	187	177	165
1¾	56⅞	252	241	234	227	221	216	202	189
2	64	284	271	263	256	249	242	227	213
12×14×1¼	44¾	197	188	183	177	173	168	158	148
1½	52½	233	222	216	210	204	199	186	174
1¾	60⅜	268	255	248	241	235	228	214	201
2	68	302	288	280	272	265	257	241	226
2¼	75⅜	335	319	310	301	292	285	268	251

WROUGHT-IRON AND STEEL COLUMNS AND STRUTS.

Owing to the many advantages of built steel columns over cast-iron columns, especially for tall buildings, and the great reduction that has taken place during the past fifteen years in the cost of steel construction, steel columns are now very extensively used in buildings, even of moderate height, and for skeleton construction, or buildings exceeding six stories in height, they are certainly much to be preferred to cast columns.

Steel trusses are also much more commonly used in buildings now than in former years, so that the architect must have at hand data for designing the same and computing the strength. In the following pages the author has endeavored to cover the subject of columns and struts quite completely, and to furnish such data as will enable one to decide upon the shape of column or strut it is best to use, and to determine the size and section with the least labor.

Forms of Steel Columns.

The forms of columns commonly used in current American building practice are those shown by the following sections:


Larimer column, 1 row of rivets.


Z-bar column, without covers, 2 rows.


4 angles and plate, 2 rows.


4-section Phœnix column, 4 rows.


Nurick column, 4 rows.

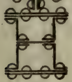

Channel column, with plates or lattice, 4 rows.


Gray column, 4 rows.


Z-bar column, with single covers, 6 rows.


Box column of plates and angles, 8 rows.


8-section Phœnix column, 8 rows.


Z-bar column, with double covers, 10 rows.

Each of these shapes has its advocates among experienced engineers, and the choice of a section is generally governed by some practical consideration, such as the cost, facility for making connections, and promptness of delivery.

Relative Advantages and Disadvantages.

The relative advantages and disadvantages of the various sections are set forth at considerable length by Mr. Joseph K. Freitag, B.S., C.E., in his very practical work on "Architectural Engineering."

In general it may be said that the factors which usually determine the choice of a section are one or more of the following points, each of which should be carefully considered when designing an important building:

1. Cost, including shop-work, availability.
2. Ability to transfer loads to centre of column, especially in cases of heavy eccentric loads.
3. Convenient connection of floor system.
4. Relation of size of section to small columns.
5. Fireproofing capabilities of the section. "Point 1 is of the greatest importance to the owner and builder, and often governs the selection of the column. Points 2, 3, and 4 are for the engineer's consideration; while point 5 is of chief interest to the architect and decorator." *

Cost, Availability.—These vary more or less at different times, the cost depending principally upon the market price of the section used, and upon the amount of shop-work required. In general it may be said that those forms which can be rolled or

* Freitag.

manufactured by any mill are likely to be the cheapest and most available, although there may often be exceptions to this rule.

Plates and angles are generally the cheapest sections of rolled steel and the most available, and the Z-bar is now being rolled by several mills. The Phœnix column is rolled only by the Phœnix Iron Company, and the Larimer column is manufactured only by Jones & Laughlins, Limited. The number of rivets required in putting the sections together, which comes under the head of "shop-work," is also an important factor in the cost of steel columns. The Larimer column possesses an advantage in this respect over all other shapes, and at the present price of steel beams this column should be one of the cheapest shapes on the market.

An objection has been found to the smaller sizes of this column, particularly the 6-inch size, that it is difficult to drive the rivets which connect the angle brackets with the I-beam flanges without interfering.

Next to the Larimer column in point of shop-work comes the Z-bar column, without cover-plates, which has two rows of rivets. For light loads this shape appears to have more advantages than any other, as it is an economical section, and the connections for floor beams and girders are quite simple, and the shape also permits of bringing the weight well into the centre of the column.

In tall buildings, however, it has almost invariably been found necessary to add cover-plates, and in some instances no less than ten rows of rivets have been required, so that for tall buildings this section does not appear to offer any advantage over columns built of plates and channels, and in point of fact it is now seldom used in high or heavy buildings. The Z-bar column, however, has been more extensively used than any other shape in the tall buildings erected during the past ten years in Chicago. Its use in Eastern cities has been far more limited. Channel columns and columns of plates and angles have also been quite extensively used both in Chicago and in the East. Although somewhat limited as to section, channel columns afford a very desirable shape, both as regards economy of material and facility for making connections. Columns built up of plates and angles present a section that can be increased to any desired area, and the area of the section can also be considerably varied without increasing the exterior dimensions.

With heavy eccentric loads it is sometimes an advantage to use

a rectangular shape, with the long axis in the direction of the eccentricity.

In practice, however, the choice of a section is generally governed more by the considerations of cost and connection facilities than by the best theoretical shape.

Further description of the different columns, and also the special advantages claimed for them, is given in the following pages.

A new type of column which has recently been patented by Mr. John Lanz, of Pittsburg, is shown by Fig. 12. The columns

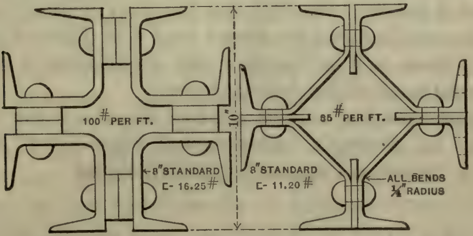


Fig. 12

are formed of rolled channel beams, bent to the necessary form and riveted together into a hollow column with projecting flanges of a T-shape. It is proposed to bend the channels so as to give columns of circular cross-section as well as the shapes illustrated. Among the merits claimed for the new column are compactness, an unusually large radius of gyration for the amount of material used, and three constant sizes of columns for 56 different sectional areas, making it very easy to get out details of the framing.

Column Connections.

This feature in column construction is a very important one, and often governs the selection of the shape to be used. Where there are only two or four beams at the same level, and these are symmetrically placed and loaded, satisfactory connections can be made for almost any of the sections, but when irregular placing of beams is necessitated, and eccentric loads must be provided for, it is important that the character of the column affords as great an opportunity as possible for the connection of plates and

angles, and for transferring eccentric loads to the centre of the column.

When wrought-iron columns were first used it was customary to use plates for connecting the story lengths, and the beams or girders often rested on these plates, as shown in Figs. 14 and 24. In the best practice at the present time these plates are often omitted, and the ends of the different lengths are closely fitted together with milled ends, and splice plates are riveted to the sides or flanges.

As it is impossible in these pages to cover the subject of column connections in anything but a general way, the only attempt that has been made in this line is to illustrate common forms of connections that have been used with different forms of columns. These will be found in the descriptions of different columns contained in the following pages.

For a more complete consideration of the subject the reader is referred to Mr. Freitag's "Architectural Engineering" and Mr. Birkmire's "Skeleton Construction in Buildings."

Number and Spacing of Rivets.

Number of Rivets Required.—No general rule can be given for the number of rivets and size of the brackets required for column connections, as the loads to be supported vary in different buildings and in different portions of the same building. The number of rivets required in each connection must therefore be determined by the rules given for designing riveted joints in Chapter XII. Connections for single beams, however, will generally require the same number of rivets as are given for beam connections, Chapter XV. The allowable strains for rivets in column connections are generally taken at 10,000 lbs. per sq. inch for single shear and 18,000 lbs. for bearing.

Spacing of Rivets.—Steel and wrought-iron columns fail either by deflecting bodily out of a straight line or by the buckling of the metal between rivets or other points of support. Both actions may take place at the same time, but if the latter occurs alone, it may be an indication that the rivet spacing or the thickness of the metal is insufficient.

The rule has been deduced from actual experiments upon wrought-iron columns that the distance between centres of rivets should not exceed, in the line of strain, sixteen times the

thickness of metal of the parts joined, and that the distance between rivets or other points of support, at right angles to the line of strain, should not exceed thirty-two times the thickness of the metal.

Z-Bar Columns.

This column was designed by Mr. C. L. Strobel, C.E., about the year 1887, and for a time the bars were rolled only by the Carnegie Steel Company. At the present time they are rolled by nearly all of the large mills, so that they can be obtained as readily as channels or angles. For buildings of moderate height and loading, no more advantageous section can be employed, while it is probably as cheap as any. It has also been used quite extensively in tall buildings, although at the present time columns built of plates and angles appear to be more generally used.

For buildings of ordinary height the column is formed of four Z-bars and one web plate, with two rows of rivets. When unusually heavy loads must be provided for, as in the case of columns for the lower stories of high buildings, the above-mentioned section may be reinforced to the required strength by using outside cover-plates, as shown on page 478, or cover-plates and angles, forming a closed or box column.

Connections.—The usual form of base plate, and the manner of supporting single beams where the column is continuous, are shown by Fig. 13. The beams should extend to within ½ inch of web plates, and should be also bolted or riveted to the supporting angles.

The usual connection of one column to another is shown by Fig. 14, which represents a plan of the cap plate, and vertical section through centre of column. The ends of the two sections should be carefully milled and connected to the cap plate by angle brackets, the whole construction being firmly riveted together. The cap plate is usually made from ½ to 1 inch in thickness, according to the load to be supported. Where beams of different depths rest on the cap plate, they may be brought to the same level by means of cast-iron bolsters.

Fig. 15 shows a detail of one of the columns used in the American Surety Building in New York, the connections shown being those of the sixteenth- and seventeenth-story floor beams. It will be noticed that in this column the end connections do not come at the floor level, but at some distance from it, and the

cap plate does not project beyond the Z-bars, the joint being secured by means of splice plates. The object in giving such a long bearing under the beams was to obtain stiffness to resist

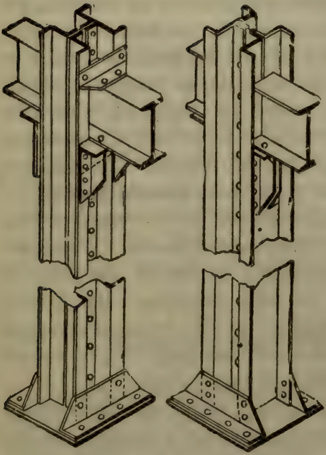


Fig. 13

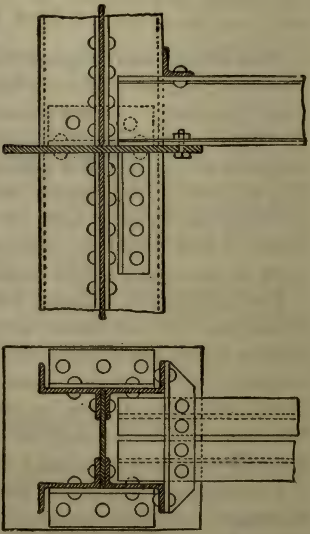


Fig. 14

the horizontal wind pressure, four rivets being placed in each side of the lower flange.

The connections shown in Fig. 15 are also applicable to channel and plate and angle columns.

The standard connections for double I-beam girders and single floor beams to Z-bar columns, detailed on page 436, were designed by the Carnegie Steel Company to fairly cover the range of ordinary practice. When the maximum loads in tons indicated for each case, are exceeded, the connections may be correspondingly strengthened by simply using longer vertical angles for the brackets and increasing the number of rivets. In proportioning these connections the shearing strain on rivets was assumed of a maximum intensity of 10,000 pounds per square inch.

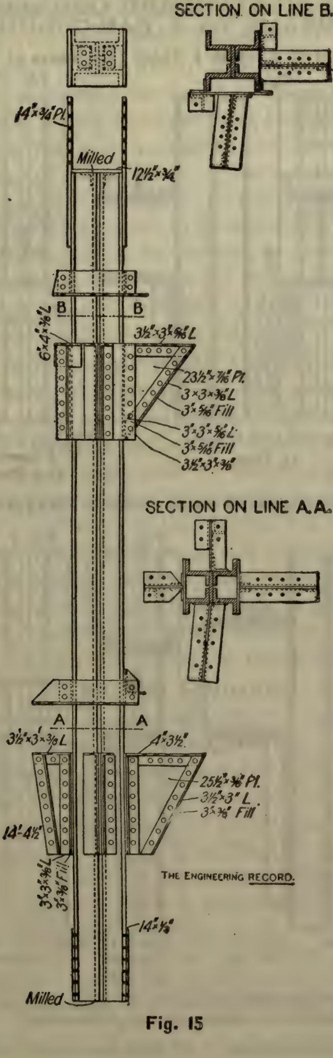
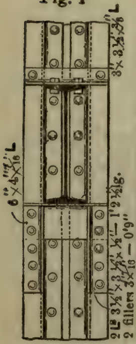

SECTION ON LINE B.B.
14"×3/4" Pl.
Milled
12½"×3/4"
B
B
6"×4"×3/8" L
3½"×3"×5/16" L
23½"×7/16" Pl.
3×3×3/8" L
3"×5/16" Fill
3"×3"×5/16" L
3"×5/16" Fill
3½"×3"×3/8"
SECTION ON LINE A.A.
A
A
3½"×3"×3/8 L
4"×3½"
25½"×3/8" Pl.
3½"×3" L
3"×3/8" Fill
14'-4½"
3"×3"×3/8" L
3"×3/8" Fill
THE ENGINEERING RECORD.
14"×½"
Milled

Fig. 15

DETAILS OF STANDARD CONNECTIONS OF I BEAMS TO Z-BAR COLUMNS.

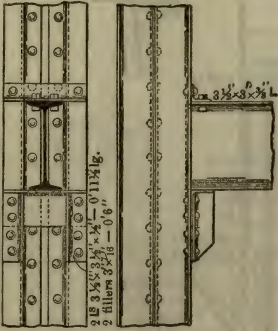
Connections of a single I Beam to Flanges of Z Bars.

Fig. 1



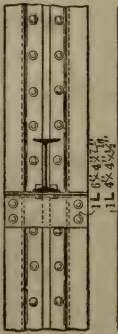
20"I Beams
44 Tons.

Fig. 2



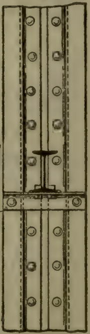
15" and 12" I Beams
35 Tons.

Fig. 3



10½," 10," 9" and 8"
I Beams
17.6 Tons.

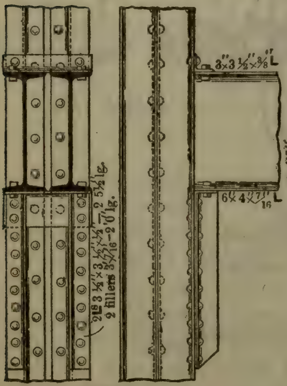
Fig. 4



7" and 6"
I Beams
8.8 Tons.

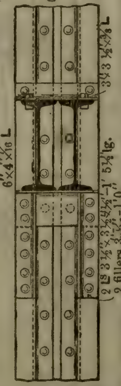
Connections of a double Beam girder to Flanges of Z Bars.

Fig. 5



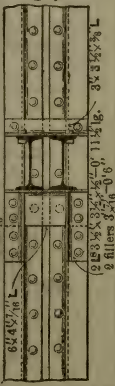
20"I Beams
88 Tons.

Fig. 6



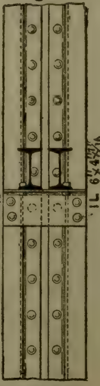
15" and 12"
I Beams
53 Tons.

Fig. 7



10½," 10," 9" and 8"
I Beams
35 Tons.

Fig. 8



7" and 6"
I Beams
17.6 Tons.

The number of tons indicated, denote the loads on single beams or girders for which the connections are proportioned.

Rivets and Bolts ¾ dia.---All Bolts have beveled heads.

The standard sections of Z-bar columns as made by the Carnegie Steel Co., together with the safe loads, are given on pages 474–481. The properties of Carnegie Z-bars are given in Chapter X.

Constant Dimension Z-bar Columns.

On page 482 is shown a section of constant dimension Z-bar columns designed by the Carnegie Steel Company, for the purpose of keeping the *same outside dimensions* throughout the successive stories of the structure. The advantage of this lies in the quicker preparation of plans and subsequent shop details; the convenience to the architect in dimensioning walls and pillars, and the simplification of the fireproofing work.

For buildings of not more than six or eight stories, however, these advantages are not sufficient to offset the disadvantage of the extra space occupied by the column.

Angle and Plate Columns.

Four angles and a plate riveted together as shown by Fig. 15*a* are now being quite extensively used in building construction, particularly for columns having an unsupported length of less than 90 radii, also for the outer posts in steel mill buildings, and for light posts supporting depot roofs, etc. Columns of this section are especially convenient for making beam and girder connections, and for splicing, and are also well adapted to resisting eccentric loads. The width of the plate is generally such that the least radius of gyration is in the direction r_2, which may be obtained directly from the tables on pages 316 and 318.

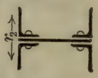


Fig. 15a

Channel Columns.

Two channels, set back to back, at such a distance that the radii of gyration will be equal about both axes, and connected by lattice bars, make a very desirable column for moderate loads, as in the upper stories, or in buildings of three or four stories in height. For greater loads, cover-plates may be riveted to the flanges in place of the lattice, as in Fig. 18. Such columns are very satisfactory, especially for making connections, provided that only a single cover-plate on each side is required. Three channels, riveted together, as in Fig. 16, also make a good column for light loads; in fact it was this combination which suggested the Z-bar column. Three I-beams riveted together in the same way have also been used for columns, but it is not an advantageous shape.

Fig. 17 shows a light lattice column, with base-plate, and Fig. 18 a typical channel column with single cover-plates, and the manner of making splices and connections.

Fig. 15b
Type of Posts used for Supporting the Boston Elevated Ry.

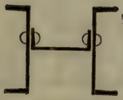


Fig. 16

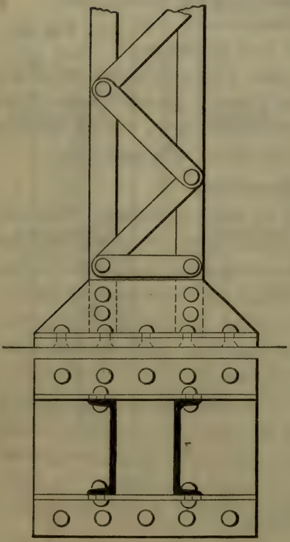


Fig. 17

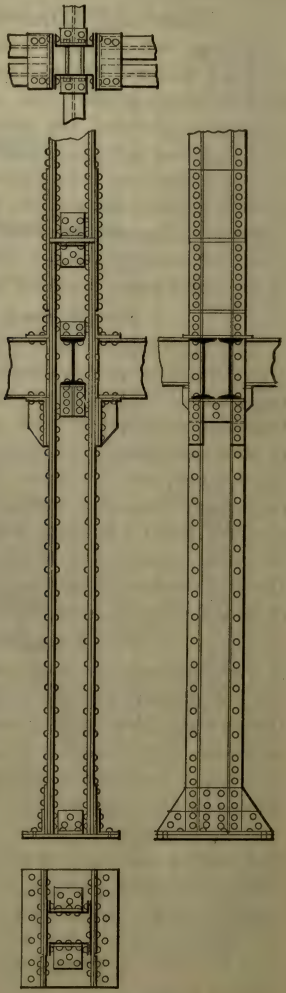


Fig. 18
Channel Column, with Cover-plates.

Rule for Latticing of Channels and Angles.—When channels are connected by latticework, as in Fig. 19, that there may not be a tendency in the channels to bend between the points of bracing, the distance l should be made to equal the total length of strut, multiplied by the least radius of gyration of a single channel, and the product divided by the least radius of gyration for the whole section; or, $l=\frac{rL}{R}$ where the letters have the following significance:

l = length between bracing;
L = total length of strut;
r = least radius of gyration for a single channel;
R = least radius of gyration for the whole section.

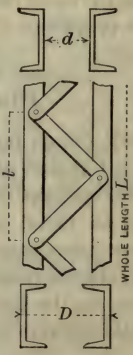


Fig. 19

This same rule will also apply for angles, though with them the latticework is generally doubled, as in Fig. 20.

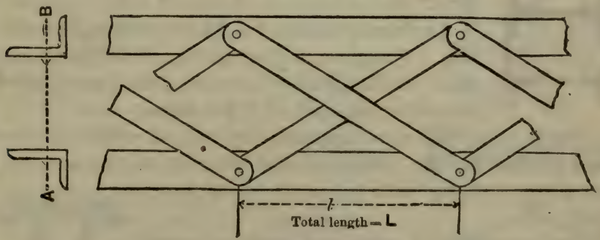


Fig. 20

Generally it is found desirable to make the distance l less than that obtained by the above formula, or so that the inclination of lattice-bars will be about 45° with the axis of the column or strut.

The size of the lacing-bars should not be less than that given in the following table:

Distance l, Fig. 20, or $\frac{1}{2} l$, Fig. 19.	Size of bar, in inches.	Distance l, Fig. 20, or $\frac{1}{2} l$, Fig. 19.	Size of bar, in inches.
less than 6″	$1\frac{1}{4}\times\frac{1}{4}$	10″ or less than 16″	$2\times\frac{3}{8}$
6″ or less than 7″	$1\frac{1}{2}\times\frac{1}{4}$	16″ or less than 20″	$2\frac{1}{4}\times\frac{7}{16}$
7″ or less than 9″	$1\frac{3}{4}\times\frac{5}{16}$	20″ or less than 24″	$2\frac{1}{2}\times\frac{1}{2}$
9″ or less than 10″	$2\times\frac{5}{16}$	20″ or above	angles

The proper distance for d or D, Fig. 19, to give a pair of channels the same radius of gyration in both directions is given under the properties of channels in Chapter X.

Plate and Angle Box Columns.

"For high buildings or heavy loads, where the required sectional area is greater than can be obtained by using Z-bar columns without cover-plates, the box column of plates and angles will be found most satisfactory. This column form possesses great advantages regarding connections, in that square surfaces are always presented. Box columns were used in the Masonic Temple, the highest building in Chicago, and in the Park Row Building, one of the highest structures in New York City." *

Fig. 21

Fig. 21 shows a heavy column section, in the Park Row Building, composed of 3 web-plates, $24'' \times \frac{11}{16}''$, 4 covers, $48'' \times \frac{11}{16}''$, and 8 angles, $6'' \times 6'' \times \frac{13}{16}''$, and designed for a load of 1,450 tons.

A column composed of 10 web-plates, 4 covers, and 12 angles, and weighing 46,980 lbs., was used in the Waldorf-Astoria Hotel to support an estimated load of 2,700 tons.

The most common form of box column is that shown by Fig. 22, the thickness and number of the web and cover-plates varying with the load to be supported.

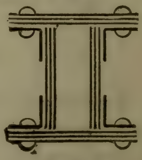


Fig. 22

Several examples of plate and angle columns are given in Chapter XXVIII.

Ordinary connections for box columns are made as shown in Figs. 15 and 18. More elaborate connections are shown in Chapter XXVIII.

The Phœnix Segmental Column.

This column has now been on the market for over thirty-eight years and has been very extensively used in buildings, and also for posts in bridges, and for piles and for wharves and piers.

---

* J. K. Freitag.

In the anthracite coal regions of Pennsylvania it is very extensively used as shafts for coal screens.

The sections were first rolled of wrought iron, and for a time in both steel and iron, but are now made only of steel.

The advantages claimed for this column by the manufacturers are: Economy of metal, simplicity of construction, adaptability to the requirements of building construction, and its cheapness.

These columns are made up of the rolled segments "C," Fig. 23, which are riveted together through the flanges with rivets about 6 inches apart. Between every two segments an iron bar is frequently inserted, through which the rivets pass. These bars, or "fillers," as they are called, increase the area of the cross-section and contribute much to the strength of the pillar. Table XX. gives the sizes of the columns rolled by the Phœnix Iron Company, as published in their book of sections, and also the radius of gyrations and safe loads.

The largest standard size of this column has a sectional area of 90.9 square inches, capable of sustaining 615 tons with an unsupported length of 36 feet.

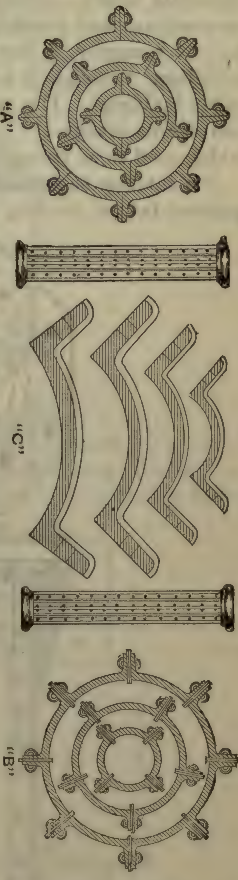


Fig 23.

The column can be made of almost any length desired. In the Schiller Theatre Building, Chicago, there are Phœnix columns 92 ft. 10 ins. long, while in the Chicago Board of Trade 12-section Phœnix columns, 3′ 3″ in diameter were employed for an unsupported length of 90 ft.

Phœnix columns are in use in several prominent high buildings, notably the "World," "Dun," and "Commercial Cable" buildings, New York; the Wainwright Building, St. Louis; Crocker

Building, San Francisco, and in a great number of buildings of moderate height. Owing to the difficulty of making elaborate connections, and possibly also to the fact that it is rolled only by one company, it has not been much used in the later high buildings, although it is still used to a considerable extent in other buildings. The sections of the Phœnix column afford a convenient means of "jacketing" cylindrical cast-iron columns which need to be strengthened.

The interior surfaces of all Phœnix columns are thoroughly painted before riveting the segments together. After twenty

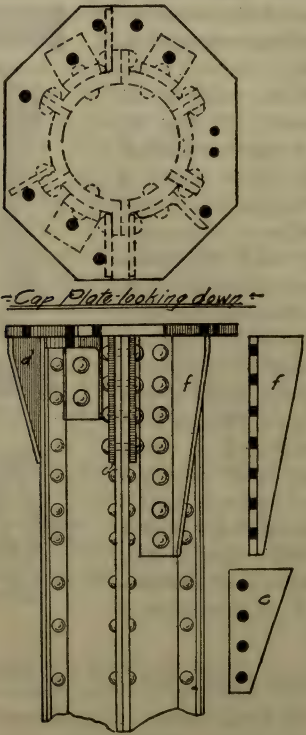


Fig. 24

years of service in exposed situations columns have been cut open and found uninjured by rust, and the paint still in good condition.

Connections.—For ordinary floor-beam connections, and for joining the ends of the columns, a connection similar to that shown in Fig. 24 has generally been used. The cap, which is usually a single plate $\frac{3}{4}$ inch to 1 inch in thickness, is made of sufficient size to give the requisite bearing for the beams or girders, and is connected to the column by means of brackets riveted to the shell

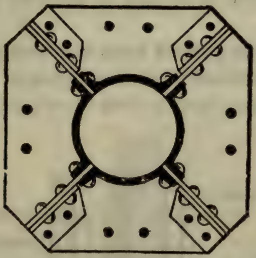


Fig. 25

and plate. The cap is also further supported by means of gusset-plates, *f* and *c*, riveted either outside the flanges or directly to the shell of the column. The upper column is set after the floor

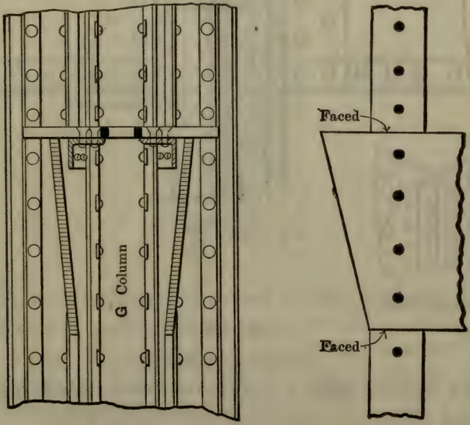


Fig. 26

system is in place, and is secured to the cap-plate by angle brackets. When columns with fillers are used the method shown in Figs. 25 and 26 is generally followed. A cap-plate is used as in

the other connection, but is supported by angles riveted to the extended fillers.

Fig. 26 shows a bracket formed beneath the cap-plate in a similar manner, a trapezoidal plate being inserted between the sections, in place of the filler, to support the bracket-plate. One of the plates passes through the column and is riveted at both sides; the other plate is riveted to the first at the centre of the column.

The latest and most perfect type of connection for this column, especially where channel or riveted girders producing eccentric loads are to be supported, is that shown in Fig. 27. A pintle,

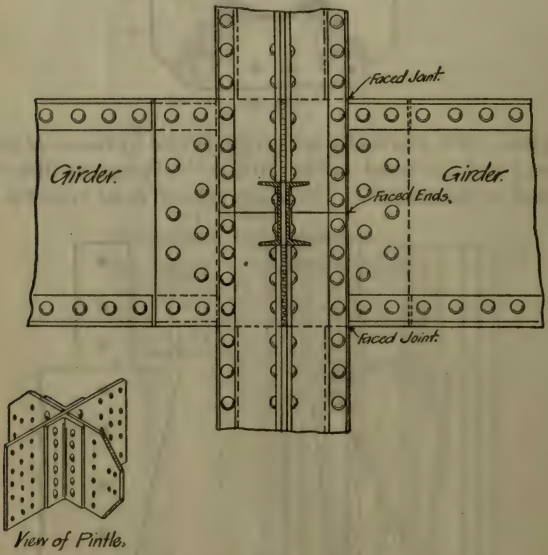


Fig. 27

Continuous Column.

shown at a reduced scale, is inserted between the flanges, and the girders and beams are riveted directly to the pintle.

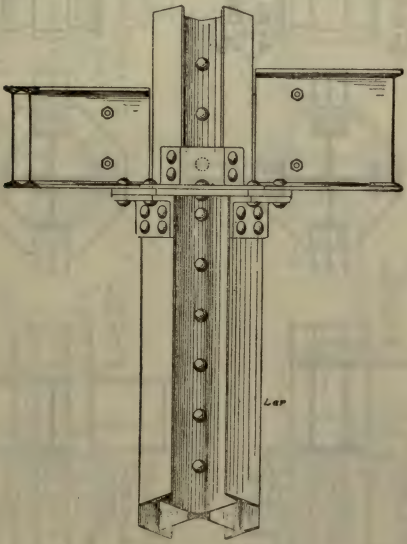
This connection is probably as nearly perfect as any that could be devised for any column, as the load is transmitted by the pintle to all parts of the column, and the pintle also greatly stiffens the column at the point of connection.

The joint of the column occurs at the centre of the beams or girders, and the column is thus made continuous from cellar to roof, with no brackets either above or below the girder.

Larimer Column.

(Manufactured by Jones & Laughlins, Limited, Pittsburg.)

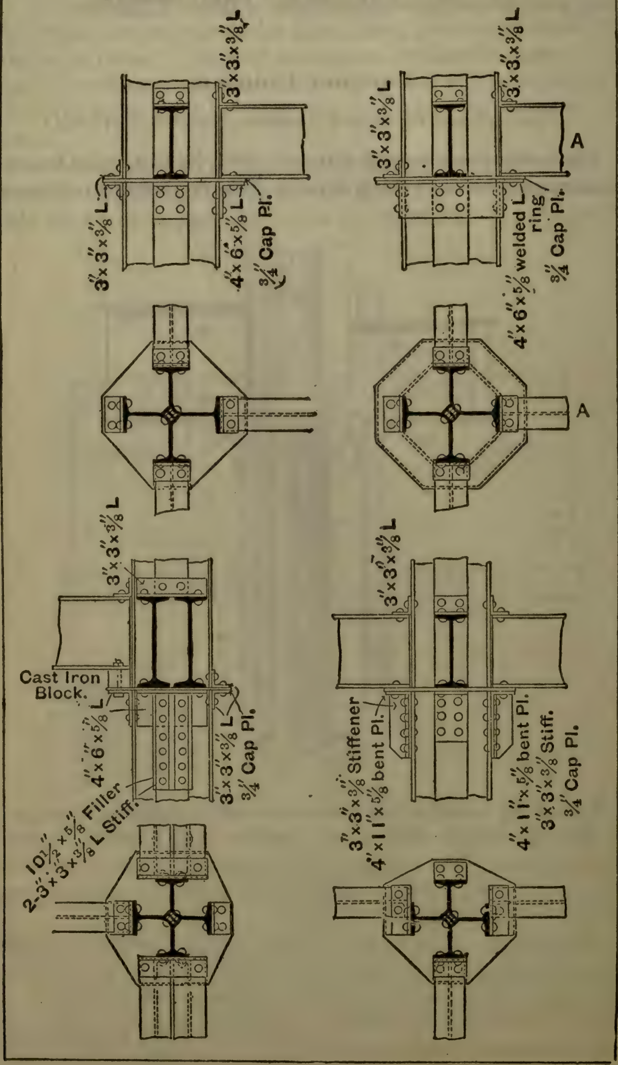
This column was patented June 2, 1891, its first use in building construction being in the Newberry Library Building in Chicago.



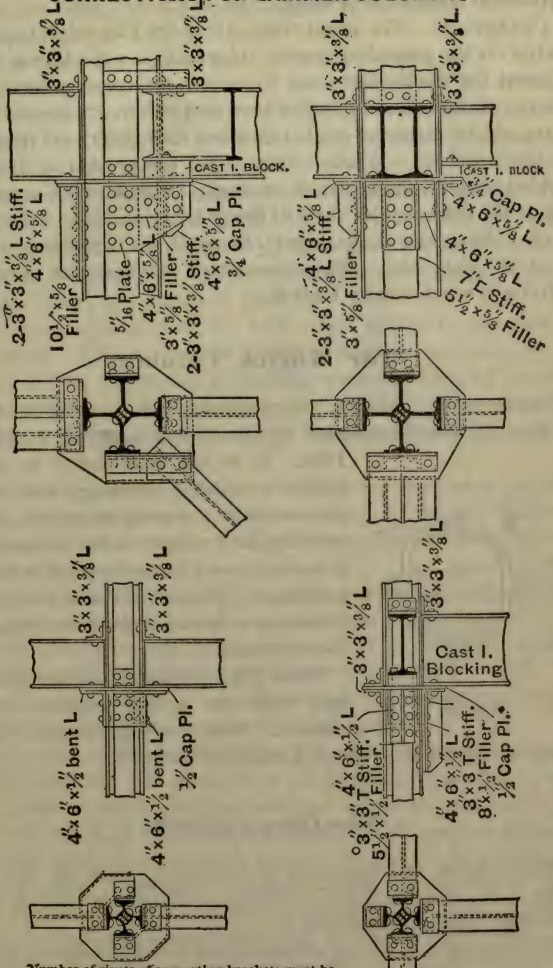
It is made by bending two I-beams at right angles in the middle of the web and riveting them together with a small I-shaped filler between. The column is very light and compact, and has but one row of rivets. Table XXI. gives the strength and dimensions of the standard sections.

The strength of the larger columns may be increased by reinforcing the flanges with steel plates, but the expense of doing this is so great that it will be cheaper to use some other section. A good many tons of this column are sold annually, but it is not very extensively used in buildings. It is much used for the

CONNECTIONS FOR LARIMER COLUMNS.



CONNECTIONS FOR LARIMER COLUMNS.



Number of rivets of supporting brackets must be determined in accordance to load by using

¾″ Rivets for all columns made of 7″ × 18, 3 lb I. B. and upward
⅝″ " " " " " " 6″ × 12, 75 lb. to 7″ × 15, 25 lb. I. Bs.
½″ " " " " " " 5″ × 16 lb. I. Bs. and less.
One ¾″ rivet may be allowed for the bending of a ¾″ Cap Plate.

support of windmills and water-tanks, where a light and inexpensive column is desired.

Connections.—The usual connections for this column are illustrated on the preceding pages. Generally a cap-plate is used to support the floor system and to receive the upper column. The flanges of the columns, both above and below, are secured to the cap-plate by means of angle brackets, with two rivets in each leg of the brackets. When a more rigid connection is desired, a welded ring, in the shape of an angle, is made to fit around the top of the column, the vertical flange being riveted to the flanges of the column and the horizontal flange to the cap-plate, as shown at *A A*, page 446. This connection is desirable when the column is eccentrically loaded.

The Nurick Column.

This column, formed of four bent channels, riveted together as in Fig. 28, was introduced by Jones & Laughlins, Limited, in 1898. It is not intended to be used in ordinary skeleton buildings, but only in places where a strong column is desired with the least weight in the column itself. It has been used to some extent in foundry buildings. Where the column is exposed it makes a better appearance than the Z-bar column.

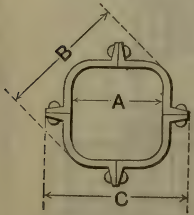


Fig. 28

Table XXII gives the dimensions and safe loads for a few sections of this column. Details of splices and connections are given in Jones & Laughlins' Manual.

The Gray Column.

This column was patented in December, 1892, by Mr. J. H. Gray, C.E., and for a time was quite extensively used in buildings of skeleton construction.*

The column is made up of angle-bars, riveted together in pairs

* This column has been used in a number of prominent buildings, but by many engineers it is not now regarded with favor, owing to the difficulty of making satisfactory connections for eccentric loading.—J. K. FREITAG.

and braced about every two feet in length by tie-plates usually 8 or 9 inches wide, riveted to the angles as shown by the section drawings, Figs. 29–32.

The special advantages claimed for this column are:

1. *A strong economical section.* As fully half of the metal is at the extreme outer edge of the column, and practically none at the centre, the radius of gyration is very large in proportion to the weight of the metal. Moreover, as angles are the cheapest shape of rolled steel that is manufactured, and are made by every rolling-mill, they can be obtained at the lowest market price, and the columns can be built by any bridge shop by paying a small royalty to the patentee. (Gray columns were made by fourteen different bridge companies in 1895.)

2. *Size of column does not vary when section is increased or diminished.* This enables the architect to vary the section of the column to suit differences in loading, without changing the outside dimensions, thus rendering the engineering work much simpler, and enabling the use of uniform sizes of fire-proof blocks.

3. *Does away with "cap-plates" and joins sections of columns firmly together, making a continuous column.* In the Reliance Building, Chicago, there is a Gray column 290 feet long, built in one piece at the shop.

4. *By varying the size or thickness of angles, and adding cover-plates, any strength that may be required can be obtained.*

5. *Has four flat sides for connections.* The usual connections for single and double beams are shown by Fig. 32.

The joint in the column should be made above the floor system, and the two portions connected by splice-plates.

Where eccentric loads are to be supported by this column it is essential that the column be very rigidly bound together by outside plates or angles opposite or just below the connection, as otherwise the load will be borne mainly by the pair of angles to which the girder is connected, and not by the whole column.

6. *Provides continuous air-space from basement to roof.* Tests made in the hydraulic machine of the Keystone Bridge Works on 14-inch square columns, 11 feet long, developed a resistance to crushing of from 38,000 to 40,000 pounds per square inch of cross-section, and a modulus of elasticity of from 24,030,000 to 27,750,000 pounds.

Table XXIII. of this chapter gives the safe loads in *thousands of pounds*, as computed by Mr. Gray, for the sizes of columns

that have been most extensively used. Experience has shown that these tables cover nearly any ordinary steel skeleton building and give all needed sections from basement to top of same.

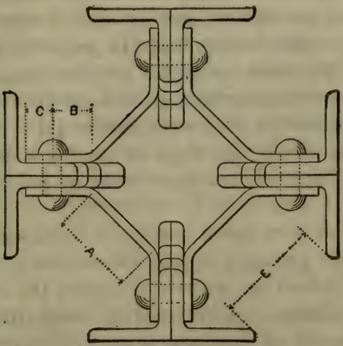


Fig. 29

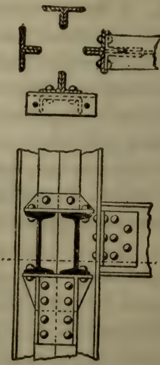


Fig. 32

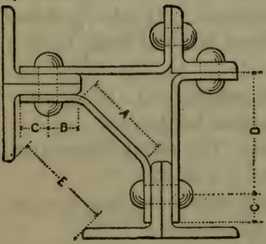


Fig. 30

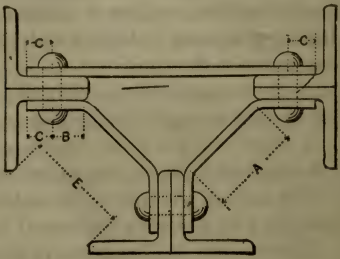


Fig. 31

Steel Struts in Trusses.

These are generally made of a pair of latticed channels, or channels and plates for heavy trusses with pin connections, and either of a pair of light channels or a pair of angles with uneven legs for light trusses. For roof trusses having a span not exceeding 80 feet a pair of $4 \times 6 \times \frac{3}{4}$ inch angles is generally sufficient for any of the compression members unless subject to transverse strain, and the minor struts are very often made of a pair of $2 \times 2\frac{1}{2} \times \frac{1}{4}$ inch angles. The angles are placed from $\frac{1}{2}$ to $\frac{3}{4}$ inch

apart, to permit the filler-plate used in the joints to go between them.

For compression members subject to transverse strain a pair of channels generally offer the best section. If necessary the channels can be reinforced by plates at top and bottom.

A pair of angles, with a deep web-plate riveted between, are often used for the principles of Fink trusses where they are subject to a slight transverse strain. See Chapter XXV.

For very light compressive stresses and short members a single angle is sometimes used. If the stress requires a greater section than that of one $3 \times 3 \times \frac{1}{2}$ inch angle, it will be cheaper and better to use a pair of smaller angles.

Where angles are used in pairs they should be *connected by a rivet* and small filler-plate every two feet in length, to prevent the angles from springing apart.

Maximum Length.—It is good practice not to use a strut whose unsupported length exceeds 150 times its least radius of gyration, or 50 times the least width of the member. For size of *lattice-bars* see page 439.

Strength of Steel and Wrought-iron Columns and Struts.

Prof. Wm. H. Burr, in his "Strength and Resistance of Materials," states that "The general principles which govern the resistance of built columns may be summed up as follows:

"The material should be disposed as far as possible from the neutral axis of the cross-section, thereby increasing the radius of gyration (r);

"There should be no initial stress;

"The individual portions of the column should be so firmly secured to each other that no relative motion can take place in order that the column may fail as a whole, thus maintaining the original value of r."

The experiments quoted by Prof. Burr would seem to indicate that a closed column is stronger than an open one, this being apparently due to the fact that the edges of the segments are mutually supporting when held in contact by a complete closure. From a theoretical standpoint, therefore, the Phœnix or Nurick shapes undoubtedly present the most favorable section for resisting compression, as they form a closed column, and the metal is

all at the outer line and equally disposed around the neutral axis. With the pintel connection, shown in Fig. 27, it would seem that these columns would have a greater ultimate resistance than open forms, such as the open Z-bar, Gray, and Larimer, although the latter column has developed a very high ultimate resistance.

It should also be remembered that any form of column having a maximum and minimum radius of gyration, such as is the case with a single I-beam, channel, or angle, is not economical for use under a single concentric load, as the minimum radius must be used in the calculation, and part of the material is to a certain extent wasted, when we consider the ideal efficiency of the column.

Formulas.

A great many different formulas have been published for the strength of wrought-iron and steel posts, and scarcely any two leading structural engineers use precisely the same formula. Previous to the year 1888 a formula similar to formula (12), and known as the Gordon formula, was generally used for all forms of columns, although with more or less variation in the constants. During the year 1888 Mr. C. L. Strobel deduced the following formula for the ultimate strength of iron Z-bar columns:

Breaking loads in lbs. per sq. in. of section area $= 46{,}000 - 125 \frac{l}{r}$.

This formula appeared to agree a little more accurately than the Gordon formula with the results of tests that had been made on full-size columns, and as it was easier of application a modification of it was adopted by the Carnegie Steel Company for computing the strength of their Z-bar columns.

This form of formula is now known as the *straight-line formula*, and as it appears to give a satisfactory reduction of loads in proportion to length of column, and is comparatively easy of application, some form of the straight-line formula is now generally used by structural engineers for steel columns and struts in preference to Gordon's formula. For the constants used in the "straight-line formula," however, there is no uniform practice, except perhaps in the case of heavy columns, for which formula (11) is quite generally used.

Gordon's formula is still used by many engineers, and as it is the standard used in the Boston Building Law it is given below as formula (12). A comparison of formulas recommended by

different engineers, and contained in building laws will be found at the end of this chapter. Those formulas which, in the opinion of the author, most nearly represent the best current practice are given below.

Formulas for Safe Loads, in Pounds, on Steel and Wrought-iron Columns and Struts.

Safe load, $W = p \times$ sectional area (in sq. inches) of column or strut. (10)

For steel columns in buildings:

$$p = 17{,}100 - 57\frac{l}{r}; \tag{11}$$

$$\text{or} \quad p = \frac{12{,}000}{1 + \dfrac{l^2}{36{,}000\,r^2}}. \tag{12}$$

For steel struts in trusses:

$$p = 13{,}500 - 50\frac{l}{r}. \tag{13}$$

For wrought-iron columns:

$$p = \frac{9{,}000}{1 + \dfrac{l^2}{36{,}000\,r^2}}; \tag{14}$$

in which l = length of column *in inches*, and r = least radius of gyration (see Chapter X.). The length of the column is measured between the points where it is supported sideways, and usually between the floor beams.

Maximum Safe Load for Columns and Struts.

For wrought-iron posts where the length in inches divided by the least radius of gyration is less than 30, p may be taken at 9,000 lbs. to the square inch.

For steel struts, in trusses, where $\frac{l}{r}$ is less than 50, p should be taken at 11,000 lbs. per square inch, unless the section is very large, when 12,000 lbs. may be used.

For steel columns, such as are used in buildings, it is customary to allow from 12,000 to 14,000 lbs. per square inch of section when the length is less than 90 radii.

When 14,000 lbs. is used for p, however, an increase in area should be made for any eccentricity in the loads. (See formula (15).)

The tables for Carnegie Z-bar columns and for the Larimer column are computed at 12,000 lbs. per square inch for lengths of 90 radii and under, and those for the Phœnix columns at 14,000 lbs. for the same ratio of $\frac{l}{r}$. The Chicago Building Ordinance specifies 13,000 lbs. for lengths of 60 radii and under. Formula (11) was used in computing the strength given in the tables for the Gray columns for all the values of $\frac{l}{r}$, and for Phœnix, Z-bar, Larimer, and channel columns exceeding 90 radii in length. This formula also very nearly corresponds with that given in the Chicago Building Ordinance.

Formula (13) is about an average of the constants used by leading structural engineers for angle and channel struts in trusses, and was used by the Passaic Rolling Mill Company for computing the safe loads for angle struts and I-beam struts published in their handbook. It is believed that the stresses permitted by this formula are such that it may be used for ordinary truss construction without allowance for rivet-holes. It may be used for either pin- or rivet-connected struts.

According to Prof. Wm. H. Burr, "the records of tests of wrought-iron channel columns with both pin and flat ends, made at the U. S. Arsenal at Watertown, Mass., have shown conclusively that the ultimate resistance of columns with flat ends will nearly invariably, if not always, fall below those of the same columns with pin ends." This is accounted for by the fact that with a pin-end column the centre of stress is practically at the centre of the section of each end, and also that the very considerable friction of the pin against the pin-hole exerts a considerable moment tending to hold the ends of the column in a "fixed" condition in a plane normal to the axis of the pin.

On the other hand, with square ends, no matter how carefully finished, it is almost impossible to apply the load so that the centre of stress will coincide with the centre of the column.

The old classification of "square," "pin and square," and "pin" has therefore been abandoned.

Application of Formulas.

EXAMPLE 1.—What is the maximum load for a steel column 12 feet long, composed of two 6-inch 8-lb. channels placed back to back, and secured by latticework?

Ans. To obtain the maximum resistance of the section the channels should be placed $3\frac{1}{2}$ ins. apart (see col. *d*, page 299). The least radius of gyration (r) will then be the same as for a single channel about axis AB, which is 2.34 (col. iv.).

To find the values of p, we will use formula (11): then $p = 17{,}100 - 57\frac{l}{r} = 17{,}100 - 57 \times \frac{144}{2.34} = 13{,}595$ lbs. Substituting this in formula (10), we have safe load for column $= 13{,}595 \times 4.76$ (area of two channels) $= 64{,}712$ lbs.

When the value of p obtained by formula (11) exceeds 12,000 lbs. it is recommended that 12,000 lbs. be used instead of the value obtained by the formula, unless the loads assumed are much in excess of the probable actual loads, or the column has a large and closed section, when 14,000 lbs. may be made the maximum.*

EXAMPLE 2.—What is the safe load for a column 16 feet long, composed of two 12-inch 30-lb. channels, with $\frac{1}{2}$-inch $\times$ 12-inch plates, riveted to channels, as shown in Fig. 33.

Ans. The first step will be to find the least radius of gyration, by means of the methods explained on page 286.

From the table of properties of channels we find that for a single channel $I_1 = 5.21$, $x = .677$, width of flange $= 3.17$, and area $= 8.82$.

The distance (d) between the backs of channels would then be $12 - (3.17 \times 2) = 5.86$. As this is less than d, in the table of properties, the least radius of gyration will be about the axis $C-D$. The distance, l (Fig. 33) $= \frac{5.86}{2} + .677 = 3.6$. Then

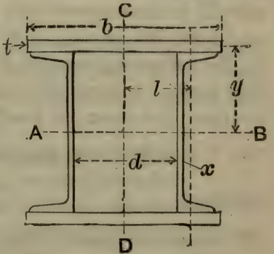


Fig. 33

* See last pages of this chapter.

the moment of inertia about CD will be (see page 286),

for the channels, $2 \times (8.82 \times (3.6)^2 + 5.21) = 239.03$

for the plates, $\dfrac{2 \times \frac{1}{2} \times 12^3}{12} = 144$

Total moment $= 383.03.$

Dividing this by the area of the section, which is 29.64, we have 12.92 for the square of the radius of gyration, and

$$r = \sqrt{12.92} = 3.6 : \frac{l}{r} = \frac{192}{3.6} = 53.3.$$

As this is less than 90, we should not use formula (11), but multiply the area of the column by the allowable strain per sq. inch, which for such a large section may safely be taken at 13,000 lbs. Then $29.64 \times 13{,}000 = 385{,}320$ lbs. = safe load.

Eccentric Loads.

Where columns are used in tiers, one above another, the beams and girders which they support must necessarily rest on brackets or pintle-plates, beyond the centre of the column.

Such methods of connection necessarily produce a moment tending to bend the column. When the *same load* is applied to opposite sides of the column, the moments produced by the loads will offset each other, and the centre of stress may be considered as coinciding with the axis. Whenever a beam, however, is attached to a column without a corresponding load on the opposite side, the load will be "eccentric," and the area of the column should be correspondingly increased, especially if strains as high as 14,000 lbs. per sq. inch are permitted.

The following formula, known as "Rankine's formula for eccentric loads," is generally used for computing the additional area required for eccentric loading:

$$\text{Additional area for eccentric load} = \frac{W \times d_1 \times d_0}{p \times r^2}, \qquad (15)$$

in which

W = eccentric load in lbs.;
d_0 = distance from centre of column to point of application;
d_1 = distance from centre of column to extreme fibre in direction in which column would bend;
r^2 = radius of gyration squared, for column used;
p = stress per square inch for $\frac{l}{r}$, Table XI.

Note.—In measuring the distance d_0 very much depends upon the form of the connection. Thus for single or double beams, where angle-brackets are used, d_0 should be measured to the centre of the bracket. In Figs. 13 and 24 it should be measured to the centre of the rivets in the beam flanges; for connections, such as shown in Figs. 15 and 27, it is generally considered sufficient to measure d_0 to the outside of the column.

EXAMPLE 1.—The *total* load on the top of a second-story column is 194,000 lbs., of which 30,000 lbs. comes from the end of a girder, without a corresponding load on the opposite side of the column. It is proposed to use a 12-inch Gray square column. What should be the section of the column, the distance to the centre of the bracket being $2\frac{1}{4}$ in. and the length of the column 16 feet?

Ans. Looking in the table giving the safe loads for a 12-inch Gray column, we find that the section given in the second line has a safe load of 195,000 lbs. for 16 ft. length, and we will therefore use that section as a basis.

For this section, $r=3.8$ and $\frac{l}{r}=\frac{192}{3.8}=50.5$; and from col. I., Table XI., we find the value of p for that ratio to be 14,220.

Then $W=30,000$; $d_0=8\frac{1}{4}$; $d_1=6$; $r^2=3.8^2=14.44$; and $p=$ 14,220.

Substituting these values in formula (15) we have—

$$\text{additional area}=\frac{30,000\times6\times8\frac{1}{4}}{14,220\times14.44}=7.23 \text{ sq. in.}$$

The area of the section used is 13.84, and adding to this 7.23 we have 21.07 as the required area, which corresponds with a section composed of eight $3\times3\frac{1}{2}\times\frac{7}{16}''$ angles; therefore this latter section should be used.

EXAMPLE 2.—What area would be required for a 12-inch Z-bar column under the same conditions as given in Example 1?

Ans. From the table on page 477 we find that the first section has a safe resistance of 128.3 tons, or considerably more than the load we wish to support. r for this section is 3.67 and $\frac{l}{r}=52.3$. The corresponding value for p would then be (col. I, Table XI.) 14,120. We will assume that the girder is supported as in Fig. 5, p. 436, so that $d_1=B$ (page 476) $=6.2$ in., and d_0 would be about 8.5 in.

We would then have—

Additional area for eccentric load $=\frac{30{,}000\times 6.2\times 8.5}{14{,}120\times(3.67)^2}=9$ sq. in.

The area required for the *total load*, considered as acting through centre of column $=\frac{w}{p}=\frac{194{,}000}{14{,}120}=13.8$ sq. in. Adding 13.8 and 9, we have 22.8 as the required area, which will necessitate using the second section given in the table.

Tables for Strength of Columns.

To lessen the labor of calculating the strength of steel columns and struts, of whatever shape, the author has computed Table XI., which gives safe values of p for lengths varying from 30 to 130 radii. For values of $\frac{l}{r}$ which give a decimal remainder one can readily interpolate between the values given. The values in this table should correspond exactly with the results obtained by using the corresponding formulas.

Table XII. gives the safe loads for gas or steam pipe used as columns. These pipes are apt to vary somewhat from the thickness published by the manufacturers, and when using them the architect should see that they have a thickness equal to that given in the table, if the full load is to be allowed. The ends of the pipe should be turned true to the axis, and fitted with cast-iron or steel plates, having the bearing planed or turned in a lath.

Tables XIII., XIV., and XVI. give the strength of standard channels and angles used as struts. Only those sizes that are most commonly used are given.

In Table XIII. the safe loads for both the minimum and the maximum radius of gyration are given. If the strut is used also as a beam, or is stayed so that it cannot bend sideways, the larger value may be used; but if free to bend in either direction, then the smaller value should be taken. If the struts are subjected to a transverse strain they should be computed as explained under the heading "Strut Beams," Chapter XV.

The tables giving the safe loads for Z-bar, Gray, Larimer, and Nurick columns were not computed by the author; they are, however, believed to be perfectly safe, provided that an increase in area is made for eccentric loads. This is especially necessary with the Gray columns, as the allowed value of p in many cases exceeds 15,000 lbs.

Application of Table XI.

This table will be found of most assistance in calculating the strength of struts in trusses and in making calculations for eccentric loading, as already illustrated.

EXAMPLE 1.—What is the safe resistance for a strut composed of two 5-inch 9-lb. standard steel channels, separated $\frac{3}{4}$ inch, and free to bend in either direction, the distance between joints being 7′ 6″?

Ans. From Table D, Chapter X., we find the least radius of gyration for this section to be 1; $\frac{l}{r}=90$; and from column III., Table XI., we find the value of p opposite 90 to be 9,000 lbs.; then the safe load = area $\times p = 5.3$ (area of two channels) $\times$ 9,000 = 47,700 lbs.

EXAMPLE 2.—What is the safe resistance of a 7-inch 15-lb. standard steel I-beam used as a strut, the length being 100 inches and the strut free to bend in either direction?

Ans. From the table giving the properties of I-beams, Chapter X., we find the least radius of gyration for this section to be 0.78, and the area 4.42; $\frac{l}{r}=\frac{100}{.78}=128.2$; and from column III., Table XI., we find, opposite 128, $p=7,100$, for 128.2 p would be about 10 lbs. less, or 7,090. Multiplying this by the area (4.42) we have 31,337 lbs. as the safe resistance of the strut.

By means of the tables and rules given in Chapter X. the area and radius of gyration of any standard section or any combination of sections may be found; and once these are obtained the strength of a strut or column may be readily computed, as in the above examples.

Proportion of Floor Loads Borne by Columns.

In tall buildings it is customary to reduce the column loads somewhat from the loads used in calculating the floor beams. This is done on the theory that it is quite impossible for the entire floor area in every story to be loaded to the maximum limit at the same time. For all buildings except warehouses it would seem to be good practice to design the columns to carry *all the dead load* and 75 per cent. of the assumed live load.

Thus if in an office building the dead load, or weight of the

floor construction, was taken at 80 lbs. and the live load at 80 lbs. per square foot, the load on the columns would be taken at $80+60=140$ lbs. per square foot times the floor area supported by the column. In some cases the reduction might even be greater, depending upon the live load assumed and the position of the column in the building, the reductions being greater in the lower stories than near the top.

The Building Law of Greater New York specifies that for buildings exceeding five stories in height the column loads shall be made up as follows: "For the roof and top floor the full live loads shall be used; for each succeeding lower floor it shall be permissible to reduce the live load by 5 per cent. until 50 per cent. of the live load is reached, when such reduced loads shall be used for all remaining floors." Column loads and the practice of leading architects in regard to proportioning columns to the loads, especially in high buildings, is discussed at considerable length by Mr. Freitag in his "Architectural Engineering."

Column Sheets.—In a high building the column loads vary to such an extent, and are made up of so many elements, that to avoid omissions and errors it is necessary to make a tabulated list of all the loads transferred through the columns to the footings.

In a building of skeleton construction the column loads will include floor and roof loads, wind loads, spandrel and pier loads, the weight of the columns themselves and their fire-proof covering, and in some cases special loads, such as tanks, vaults, safes, and elevator loads.

In tabulating the floor loads it is a good idea to separate the dead and live loads for convenience in proportioning the footings. See Chapter II.

Formulas for computing the wind loads on columns are given in Chapter XXVIII.; these loads are also considered as live loads.

In buildings not exceeding 100 feet in height wind loads are generally disregarded.

Eccentric loads should always be tabulated separate from the column loads.

On the opposite page is shown a form of column sheet which combines all ordinary requirements.

The total load for each story will be the sum of all of the loads above.

The table on page 462, taken from Freitag's "Architectural Engineering," shows a very convenient form of schedule for column lengths and column materials.

FORM OF COLUMN SHEET.

Story		Column No. 1.		Column 2.
		Load on column concentric	Load on column eccentric	
18th (top) Story.	Roof and ceiling, dead load			
	" " " live load			
	Masonry piers			
	Spandrels, cornice, etc.			
	Elevators			
	Tanks			
	Column and casing			
	Wind			
	Total			
	Sectional area required	sq. ins.	sq. ins.	
17th Story	From column above.*			
	Floor, dead load			
	" live load			
	Masonry piers			
	Spandrels			
	Safes, vaults, etc.			
	Column and casing			
	Wind			
	Total			
	Sectional area required	sq. ins.	sq ins.	
Basement.	From column above.*			
	Floor, dead load			
	" live load			
	Masonry piers			
	Spandrels			
	Sidewalk			
	Column and casing			
	Wind			
	Total			
	Sectional area required	sq. ins.	sq. ins.	
Footings.	Deduct ($\frac{1}{2}$) live load			
	Total footing load			
	Area of footing required	sq. ft.		

* In bringing down the loads from the column above the eccentric loads may be added to the concentric loads and their sum placed in the first column.

SCHEDULE OF COLUMN LENGTHS AND MATERIAL.

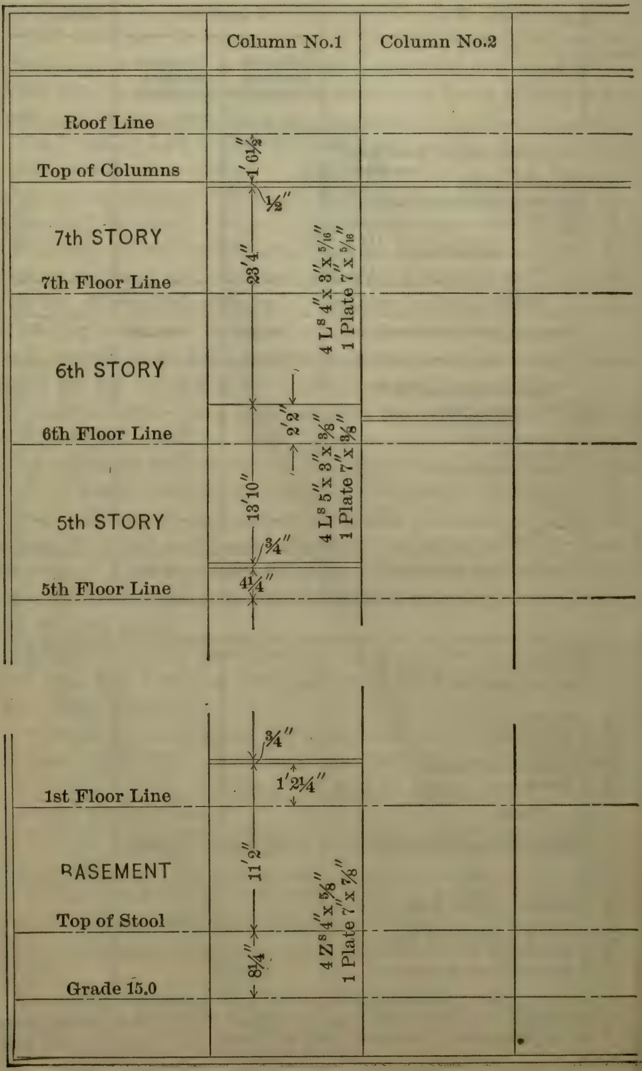


TABLE XI.—SAFE LOADS PER SQUARE INCH OF METAL AREA FOR STEEL AND WROUGHT-IRON COLUMNS AND STRUTS.

$\frac{l}{r}$ both in inches.	Steel columns. $17{,}100 - 57\frac{l}{r}$	Steel columns. $\frac{12{,}000}{1+\frac{l^2}{36{,}000r^2}}$	Steel struts. $13{,}500 - 50\frac{l}{r}$	Wrought iron. $\frac{9{,}000}{1+\frac{l^2}{36{,}000r^2}}$
	I.	II.	III.	IV.
30	....	11,706	12,000	8,780
36	....	11,581	11,700	8,686
40	....	11,488	11,500	8,616
44	....	11,388	11,300	8,541
48	....	11,277	11,100	8,458
50	14,250	11,220	11,000	8,415
52	14,136	11,161	10,900	8,371
54	14,022	11,100	10,800	8,325
56	13,908	11,040	10,700	8,280
58	13,794	10,978	10,600	8,234
60	13,680	10,908	10,500	8,181
61	13,623	10,874	10,450	8,156
62	13,566	10,841	10,400	8,131
63	13,509	10,808	10,350	8,106
64	13,452	10,772	10,300	8,079
65	13,395	10,740	10,250	8,055
66	13,338	10,704	10,200	8,028
67	13,281	10,666	10,150	8,000
68	13,224	10,633	10,100	7,975
69	13,167	10,597	10,050	7,948
70	13,110	10,561	10,000	7,921
71	13,053	10,525	9,950	7,894
72	12,996	10,489	9,900	7,867
73	12,939	10,452	9,850	7,839
74	12,882	10,416	9,800	7,812
75	12,825	10,380	9,750	7,785
76	12,768	10,341	9,700	7,756
77	12,711	10,302	9,650	7,727
78	12,654	10,264	9,600	7,698
79	12,597	10,229	9,550	7,672
80	12,540	10,190	9,500	7,642
81	12,483	10,152	9,450	7,614
82	12,426	10,112	9,400	7,584
83	12,369	10,072	9,350	7,554
84	12,312	10,033	9,300	7,525
85	12,255	9,993	9,250	7,495
86	12,198	9,954	9,200	7,466
87	12,141	9,916	9,150	7,438
88	12,084	9,876	9,100	7,407
89	12,027	9,836	9,050	7,377

TABLE XI. (*continued*).

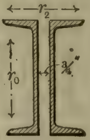
$\frac{l}{r}$ both in inches.	Steel columns. $17{,}100-57\frac{l}{r}$	Steel columns. $\dfrac{12{,}000}{1+\dfrac{l^2}{36{,}000r^2}}$	Steel struts. $13{,}500-50\frac{l}{r}$	Wrought iron. $\dfrac{9{,}000}{1+\dfrac{l^2}{36{,}000r^2}}$
	I.	II.	III.	IV.
90	11,970	9,796	9,000	7,347
91	11,913	9,756	8,950	7,317
92	11,856	9,716	8,900	7,287
93	11,799	9,677	8,850	7,258
94	11,742	9,638	8,800	7,229
95	11,685	9,600	8,750	7,200
96	11,628	9,553	8,700	7,165
97	11,571	9,516	8,650	7,137
98	11,514	9,478	8,600	7,109
99	11,457	9,433	8,550	7,075
100	11,400	9,389	8,500	7,042
101	11,343	9,352	8,450	7,014
102	11,286	9,309	8,400	6,982
103	11,229	9,273	8,350	6,955
104	11,162	9,230	8,300	6,923
105	11,105	9,188	8,250	6,891
106	11,048	9,146	8,200	6,860
107	10,991	9,104	8,150	6,828
108	10,934	9,064	8,100	6,798
109	10,887	9,022	8,050	6,767
110	10,830	8,981	8,000	6,736
111	10,773	8,941	7,950	6,706
112	10,716	8,900	7,900	6,676
113	10,659	8,860	7,850	6,646
114	10,602	8,816	7,800	6,612
115	10,545	8,776	7,750	6,582
116	10,488	8,734	7,700	6,551
117	10,431	8,694	7,650	6,521
118	10,374	8,654	7,600	6,491
119	10,317	8,613	7,550	6,460
120	10,260	8,572	7,500	6,429
121	10,203	8,532	7,450	6,401
122	10,146	8,492	7,400	6,369
123	10,089	8,451	7,350	6,338
124	10,032	8,410	7,300	6,307
125	9,975	8,368	7,250	6,276
126	9,918	8,326	7,200	6,245
127	9,861	8,286	7,150	6,215
128	9,804	8,246	7,100	6,185
129	9,747	8,206	7,050	6,155
130	9,690	8,162	7,000	6,122

TABLE XII.—SAFE LOADS IN TONS FOR GAS- OR STEAM-PIPE COLUMNS.

Computed by formula: $p = 11{,}000 - 35\frac{l}{r}$.

	Nominal size.	External diameter.	Thickness.	Weight per foot.	Area of section.	Radius of gyration.	Length in feet.				
							8	9	10	12	14
	inches.	inches.	inches.	lbs.							
Standard pipe.	2½	2.875	.204	5.74	1.59	.94	5.90	5.51	5.21		
	3	3.5	.217	7.54	2.26	1.16	9.14	8.75	8.35	7.52	
	3½	4.0	.226	9.00	2.59	1.35	11.02	10.66	10.25	9.39	8.62
	4	4.5	.237	10.66	3.33	1.50	14.45	14.11	13.65	12.72	11.78
	4½	5.0	.247	12.34	3.73	1.68	16.78	16.33	15.88	14.90	13.98
	5	5.563	.259	14.50	4.17	1.88	18.76	18.76	18.26	17.31	16.26
	6	6.625	.280	18.76	5.57	2.25	25.06	25.06	25.06	24.39	23.32
	7	7.625	.301	23.37	7.18	2.59	32.31	32.31	32.31	32.31	31.32
	8	8.625	.322	28.18	8.14	2.94	36.63	36.63	36.63	36.63	36.63
XX Strong.	2½	2.875	.56	13.68	4.09	0.82	14.10	13.04	11.86		
	3	3.5	.608	18.56	5.52	1.02	21.25	20.12	19.04	16.56	
	3½	4.0	.642	22.75	6.63	1.20	27.18	26.02	24.86	22.54	19.89
	4	4.50	.682	27.48	8.33	1.35	35.31	34.15	32.84	30.19	27.57
	5	5.563	.75	38.12	11.73	1.70	52.78	51.37	49.94	47.06	44.16
	6	6.625	.875	53.11	15.80	2.04	71.10	71.10	70.58	66.99	64.22

TABLE XIII.—SAFE LOADS IN TONS FOR STRUTS FORMED OF A PAIR OF CHANNELS.



Distance between webs, ¾ inch.

If strut is free to bend in either direction use smaller value.

Strains per square inch:

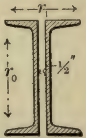
12,000 lbs. for lengths of 30 radii and under;

$13{,}500 - 50\frac{l}{r}$ for lengths over 30 radii.

D'pth in ins.	Weight per foot, lbs.*	Thick-ness of web.	Area of two chan-nels.	r_2 / r_0	Length in feet. 8	9	10	11	12	14
15	33	0.40	19.80	1.48	101.57	97.56	93.55	89.54	85.48	77.44
				5.62	118.80	118.80	118.80	118.80	118.80	118.80
	35	0.43	20.58	1.47	105.32	101.13	96.93	92.73	88.54	80.09
				5.58	123.48	123.48	123.48	123.48	123.48	123.00
	40	0.52	23.52	1.46	120.13	115.30	110.48	103.66	100.78	91.14
				5.43	141.12	141.12	141.12	141.12	141.12	140.41
	45	0.62	26.48	1.45	134.91	129.48	123.99	118.50	113.00	102.08
				5.32	158.88	158.88	158.88	158.88	158.88	157.82
	50	0.72	29.42	1.46	150.36	144.23	138.20	132.17	126.06	114.00
				5.23	176.52	176.52	176.52	176.52	176.52	174.75
	55	0.82	32.36	1.47	165.60	159.00	152.40	145.78	139.22	126.10
				5.16	194.16	194.16	194.16	194.16	194.16	192.00
12	20½	0.28	12.06	1.34	59.81	57.10	54.40	51.70	49.02	43.62
				4.61	72.36	72.36	72.36	72.36	71.99	70.43
	25	0.39	14.70	1.31	72.32	68.95	65.60	62.21	58.83	52.03
				4.43	88.20	88.20	88.20	88.20	87.28	85.26
	30	0.51	17.64	1.30	86.52	82.46	78.36	74.30	70.25	62.09
				4.28	105.84	105.84	105.84	105.48	104.25	101.78
	35	0.64	20.58	1.31	101.25	96.52	91.78	87.10	82.37	72.90
				4.17	123.48	123.48	123.48	122.65	121.16	118.33
	40	0.76	23.52	1.32	116.01	110.66	105.31	99.96	94.66	83.96
				4.09	141.12	141.12	141.12	139.82	138.06	134.65
10	15	0.24	8.92	1.24	42.94	40.78	38.64	36.48	34.32	30.01
				3.87	53.52	53.52	53.29	52.62	51.91	50.43
	20	0.38	11.76	1.20	55.86	52.92	49.98	47.04	44.10	38.22
				3.66	70.56	70.56	69.73	68.79	67.82	65.85
	25	0.53	14.70	1.20	69.82	66.15	62.47	58.80	55.12	47.77
				3.52	88.20	87.94	86.69	85.44	84.19	81.69
	30	0.68	17.64	1.22	84.40	80.04	75.71	71.35	67.03	58.34
				3.42	105.84	105.13	103.63	102.04	100.20	97.41
	35	0.82	20.58	1.26	99.76	94.82	89.93	85.04	80.16	70.33
				3.35	123.48	122.34	120.49	118.64	116.79	113.13
9	13¼	0.23	7.78	1.19	36.83	34.87	32.91	30.94	28.98	25.07
				3.49	46.68	46.48	45.82	45.16	44.50	43.15
	15	0.29	8.82	1.17	41.45	39.18	36.93	34.66	32.41	27.89
				3.40	52.92	52.52	51.81	50.98	50.10	48.64
	20	0.45	11.76	1.15	54.85	51.77	48.71	45.65	42.57	36.42
				3.21	70.56	69.50	68.38	67.29	66.20	64.00
	25	0.62	14.70	1.17	69.09	65.31	61.55	57.77	54.00	46.48
				3.10	87.83	86.43	85.00	83.56	82.17	79.30

* Of single channel.

TABLE XIII.—SAFE LOADS IN TONS FOR STRUTS FORMED OF A PAIR OF CHANNELS (*continued*).



Distance between webs, ½ inch.

If strut is free to bend in either direction use smaller value.

Strains per square inch:

11,000 lbs. for lengths of 50 radii and under;

$13,500 - 50\frac{l}{r}$ for lengths over 50 radii.

D'pth in ins.	Weight per foot, lbs.*	Thickness of web.	Area of two channels.	r_1 / r_0	Length in feet. 6	7	8	9	10	11
8	11.25	0.22	6.70	1.04	33.63	31.70	29.76	27.83	25.91	23.96
				3.11	36.85	36.85	36.85	36.85	36.85	36.85
	13.75	0.31	8.08	1.04	40.56	38.23	35.89	33.57	31.24	28.90
				2.98	44.44	44.44	44.44	44.44	44.44	44.44
	16.25	0.40	9.56	1.03	47.82	45.05	42.25	39.48	36.68	33.91
				2.89	52.58	52.58	52.58	52.58	52.58	52.58
	18.75	0.49	11.02	1.03	55.12	51.93	48.70	45.51	42.29	39.09
				2.82	60.61	60.61	60.61	60.61	60.61	60.61
	21.25	0.58	12.50	1.03	62.53	58.90	55.25	51.62	47.96	44.34
				2.77	68.75	68.75	68.75	68.75	68.75	68.75
7	9.75	0.21	5.70	0.99	28.11	26.39	24.66	22.94	21.20	19.47
				2.72	31.35	31.35	31.35	31.35	31.35	31.35
	12.25	0.32	7.20	0.99	35.51	33.33	31.15	28.98	26.78	24.60
				2.59	39.60	39.60	39.60	39.60	39.60	39.42
	14.75	0.42	8.68	0.99	42.71	40.18	37.56	34.93	32.28	29.66
				2.50	47.74	47.74	47.74	47.74	47.74	47.13
	17.25	0.53	10.14	1.00	50.19	47.15	44.10	41.06	38.02	34.98
				2.44	55.77	55.77	55.77	55.77	55.77	54.73
	19.75	0.63	11.62	1.00	57.52	54.03	50.54	47.06	43.57	40.08
				2.39	63.91	63.91	63.91	63.91	63.91	62.39
6	8.00	0.20	4.76	0.94	23.02	21.50	19.98	18.46	16.94	15.42
				2.34	26.18	26.18	26.18	26.18	26.02	25.41
	10.50	0.32	6.18	0.94	29.89	27.91	25.94	23.97	22.00	20.02
				2.21	33.99	33.99	33.99	33.99	33.32	32.48
	13.00	0.44	7.64	0.95	37.11	34.68	32.27	29.87	27.44	25.04
				2.13	42.02	42.02	42.02	41.88	40.81	39.72
	15.50	0.56	9.12	0.95	44.30	41.40	38.53	35.66	32.78	29.89
				2.07	50.16	50.16	50.16	49.68	48.33	47.03
5	6.50	0.19	3.90	0.89	18.43	17.13	15.81	14.49	13.18	11.86
				1.95	21.45	21.45	21.45	20.92	20.32	19.72
	9.00	0.33	5.30	0.90	25.17	23.41	21.65	19.87	18.11	16.35
				1.83	29.15	29.15	28.83	27.97	27.10	26.22
	11.50	0.48	6.76	0.91	32.26	30.03	27.81	25.58	23.35	21.12
				1.75	37.18	37.18	36.36	35.20	34.03	32.88
4	5.25	0.18	3.10	0.84	14.28	13.17	12.07	10.96	9.85	
				1.56	17 05	16.75	16.15	15.55	14.96	14.36
	6.25	0.25	3.68	0.84	16.95	15.64	14.33	13.02	11.70	
				1.51	20.24	19.72	18.99	18.26	17.53	16.80
	7.25	0.32	4.26	0.84	19.62	18.10	16.59	15.07	13.54	
				1.46	23.43	22.63	21.75	20.87	19.98	19.12

* Of single channel.

TABLE XIV.—SAFE LOADS IN TONS FOR SINGLE ANGLE STRUTS (STEEL).

A. ANGLES WITH UNEQUAL LEGS.

Strains per square inch:

11,000 lbs. for lengths of 50 radii and under;

$13{,}500-\frac{50l}{r}$ for lengths over 50 radii.

Size.	Thickness.	r Axis. EF.*	Area.	Length in feet.						
				4	5	6	7	8	9	10
6 ×4	3/8	0.88	3.61							
	7/8	0.86	7.99	42.78	40.00	37.21	34.44	31.64	28.86	26.07
5 ×3½	3/8	0.76	3.05							
	3/4	0.75	5.81							
5 ×3	5/16	0.66	2.40							
	3/4	0.64	5.44							
4½×3	5/16	0.66	2.25							
	3/4	0.64	5.06	24.66	22.29	19.92	17.55	15.18		
4 ×3½	5/16	0.73	2.25	11.49	10.57	9.65	8.72	7.79	6.86	
	3/4	0.72	5.06	25.73	23.62	21.51	19.40	17.29	15.18	
4 ×3	5/16	0.65	2.09	10.25	9.28	8.32	7.36	6.39		
	3/4	0.64	4.69	22.86	20.67	18.47	16.27	14.07		
3½×3	5/16	0.63	1.93	9.35	8.43	7.51	6.59			
	3/8	0.62	2.30	11.07	9.96	8.84	7.74			
	5/8	0.62	3.67	17.67	15.90	14.12	12.35			
3½×2½	1/4	0.54	1.44	6.52	5.72	4.92				
	3/8	0.54	2.11	9.55	8.38	7.21				
	1/2	0.53	2.75	12.34	10.78	9.22				
3 ×2½	1/4	0.53	1.31	5.88	5.13	4.39				
	3/8	0.52	1.92	8.52	7.42	6.31				
	1/2	0.52	2.50	11.10	9.66	8.22				
3 ×2	1/4	0.43	1.19	4.71	3.88					
	3/8	0.43	1.73	6.85	5.64					
	1/2	0.43	2.25	8.91	7.34					
2½×2	1/4	0.42	1.06	4.13	3.37					
	3/8	0.42	1.55	6.03	4.93					
	1/2	0.42	2.00	7.79	6.36					

* Axis diagonal, see p. 304.

TABLE XIV.—SAFE LOADS IN TONS FOR SINGLE ANGLE STRUTS (*continued*).

B. Angles with Equal Legs.

Strains per square inch:

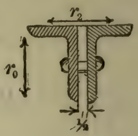
11,000 lbs. for lengths of 50 radii and under;

$13{,}500 - \frac{50l}{r}$ for lengths over 50 radii.

Size.	Thick-ness.	r Axis EF.*	Area.	Length in feet.						
				4	5	6	7	8	9	10
6 × 6	3/8	1.19	4.36	23.98	23.93	22.83	21.74	20.64	19.54	18.44
	5/8	1.18	7.11	39.10	38.96	37.14	35.35	33.54	31.72	29.93
	7/8	1.17	9.74	53.57	53.27	50.77	48.28	45.77	43.26	40.78
5 × 5	3/8	0.99	3.61	19.85	18.89	17.80	16.71	15.64	14.53	13.42
	5/8	0.97	5.86	32.23	30.50	28.68	26.86	25.06	23.24	21.43
	7/8	0.96	7.99	43.94	41.44	38.95	36.45	33.95	31.46	28.96
4 × 4	3/8	0.79	2.86	14.96	13.88	12.79	11.71	10.61	9.53	
	1/2	0.78	3.75	19.54	18.10	16.65	15.22	13.78	12.33	
	5/8	0.77	4.61	23.93	22.13	20.33	18.55	16.75	14.95	
	3/4	0.77	5.44	28.24	26.12	23.99	21.89	19.77	17.65	
3½ × 3½	5/16	0.69	2.09	10.47	9.56	8.65	7.74	6.83		
	1/2	0.68	3.25	16.20	14.77	13.34	11.90	10.47		
	5/8	0.67	3.98	19.74	17.95	16.17	14.39	12.61		
	3/4	0.67	4.69	13.26	21.16	19.06	16.96	14.86		
3 × 3	1/4	0.59	1.44	6.79	6.06	5.32	4.59			
	3/8	0.58	2.11	9.88	8.78	7.69	6.60			
	1/2	0.58	2.75	12.87	11.45	10.03	8.60			
	5/8	0.57	3.36	15.60	13.84	12.07	10.30			
2½ × 2½	3/16	0.49	0.90	3.87	3.32	2.76				
	1/4	0.49	1.19	5.10	4.39	3.66				
	3/8	0.48	1.73	7.35	6.27	5.19				
	1/2	0.47	2.25	9.44	8.01	6.57				
2¼ × 2¼	3/16	0.44	0.81	3.26	2.70					
	1/4	0.44	1.06	4.26	3.54	2.80				
	3/8	0.43	1.55	6.13	5.05	3.95				
	7/16	0.43	1.78	7.14	5.80	4.53				
2 × 2	3/16	0.40	0.72	2.70	2.16					
	1/4	0.39	0.94	3.45	2.72	2.00				

* Axis diagonal, see p. 309.

TABLE XV.—SAFE LOADS IN TONS FOR TWO ANGLE STRUTS.



LONG LEGS PARALLEL AND ½ INCH APART.

Strains per square inch:

11,000 lbs. for lengths of 50 radii and under;

$13{,}500 - \frac{50l}{r}$ for lengths over 50 radii.

Size.	Thickness.	Least r	Area of two angles.	Length in feet. 5	6	7	8	10	11	12
8 ×6	1/2	2.49	13.52	74.36	74.36	74.36	74.36	74.36	73.34	71.72
	1	2.65	26.82	147.51	147.51	147.51	147.51	147.51	147.51	144.62
6 ×4	3/8	1.67	7.22	39.71	39.71	39.65	38.35	35.77	34.47	33.14
	13/16	1.74	14.94	82.17	82.17	82.17	80.26	75.07	72.50	69.91
6 ×3½	3/8	1.43	6.84	37.62	37.57	36.13	34.69	31.82	30.38	29.00
	1/2	1.46	9.00	49.50	49.50	47.81	45.97	42.27	40.41	38.56
	5/8	1.49	11.10	61.05	61.05	59.27	57.05	52.51	50.29	48.07
	13/16	1.52	14.12	77.66	77.66	75.82	73.03	67.42	64.65	61.88
5 ×4	3/8	1.59	6.46	35.53	35.53	35.07	33.86	31.41	30.20	28.99
	3/4	1.54	12.38	68.09	68.09	66.69	64.28	59.45	57.04	54.62
5 ×3½	3/8	1.51	6.10	33.55	33.55	32.70	31.49	29.05	27.84	26.64
	3/4	1.55	11.62	63.91	63.91	62.69	60.45	55.95	53.71	51.44
5 ×3	3/8	1.27	5.72	31.46	30.50	29.15	27.80	25.09	23.75	22.39
	1/2	1.30	7.50	41.25	40.23	38.51	36.78	33.32	31.59	29.85
	5/8	1.33	9.22	50.71	49.76	47.69	45.59	41.40	39.30	37.29
	3/4	1.36	10.88	59.84	58.94	56.63	54.23	49.42	47.05	44.66
4 ×3½	3/8	1.25	5.34	29.37	28.35	27.07	25.79	23.23	21.94	20.66
	3/4	1.20	10.12	55.66	53.13	50.60	48.07	43.01	40.48	39.95
4 ×3	3/8	1.26	4.96	27.28	26.28	25.12	24.00	21.64	20.49	19.30
	3/4	1.22	9.38	51.59	49.47	47.18	44.85	40.24	37.94	35.64
3½×2½	1/4	1.12	2.88	15.58	14.81	14.03	13.26	11.72	10.95	10.18
	3/8	1.10	4.22	22.73	21.59	20.42	19.28	16.97	15.82	14.67
	1/2	1.09	5.50	29.56	28.05	26.53	25.02	21.98	20.47	18.96
	11/16	1.06	7.30	38.92	36.86	34.78	32.74	28.58	26.51	24.43
3 ×2	1/4	0.93	2.38	12.22	11.45	10.69	9.92	8.39	7.62	
	1/2	0.92	4.50	23.04	21.58	20.10	18.64	15.70	14.23	
2½×2	3/16	0.79	1.62	7.86	7.24	6.63	6.01			
	1/2	0.75	4.00	19.00	17.40	15.80	14.20			
2 ×2	3/16	0.62	1.44	6.22	5.54	4.82	4.13			
2 ×2	1/4	0.61	1.88	8.08	7.14	6.20	5.26			

TWO ANGLE STRUTS OF ┘┌ SECTION, p. 471.

2½×2½	3/16	1.07	1.80	9.63	9.13	8.64	8.10	7.11	6.61	6.12
	1/4	1.11	2.38	12.85	12.19	11 54	10.88	9.63	8.98	8.63
2¼×2¼	3/16	1.03	1.62	8.58	8.10	7.65	7.16	6.23	5.75	5.26
	1/4	1.01	2.12	11.13	10.54	9.91	9.27	8.00	7.42	6.78
2 ×2	3/16	0.93	1.44	7.37	6.94	6.48	6.01	5.07	4.60	4.13
	1/4	0.94	1.88	9.68	9.07	8.46	7.89	6.67	6.11	5.49

TABLE XVI.—SAFE LOADS IN TONS FOR STANDARD STEEL BEAMS USED AS COLUMNS OR STRUTS.

Strains per square inch: $13{,}500 - 50\frac{l}{r}$.

BEAMS UNSUPPORTED SIDEWAYS.

Size, ins.	Weight, lbs.	r	Area of section.	Length in feet.					
				9	10	11	12	13	14
15″	42.00	1.08	12.48	53.04	49.57	46.11	42.65	39.19	
	50.00	1.04	14.71	61.12	56.85	52.62	48.38	44.13	
	60.00	1.21	17.67	79.87	75.45	71.12	66.70	62.33	57.95
12″	31.50	1.01	9.26	37.76	35.00	32.24	29.50		
	35.00	0.99	10.29	41.39	38.28	35.16	32.05		
	40.00	1.08	11.84	50.32	47.03	43.75	40.46	37.18	
10″	25.00	0.97	7 37	29.24	26.95	24.67	22.40		
	40.00	0.90	11.76	44.10	40.19	36.28			
9″	21.00	0 90	6.31	23.66	21.56	19.46			
	35.00	0.84	10.29	36.40	32.72				
8″	18.00	0.84	5.33	18.85	16.95				
	25.50	0.80	7.50	25.31	22.50				
BEAMS SUPPORTED SIDEWAYS.									
8	18.00	3.27	5.33	31.58	31.08	30.59	30.11	29.62	29.14
	25.50	3.02	7.50	43.93	43.17	42.43	41.68	40.95	40.20
7	15.00	2.86	4.42	25.67	25.20	24.74	24.27	23.81	23.35
	20.00	2.68	5.88	33.76	33.10	32.45	31.79	31.13	30.47
6	12.25	2.46	3.61	20.40	19.96	19.53	19.09	18.64	18.20
	14.75	2.35	4.34	24.31	23.76	23.21	22.65	22.09	21.53
	17.25	2.27	5.07	28.19	27.52	26.86	26.18	25.51	24.84
5	9.75	2.05	2.87	15.59	15.17	14.75	14.33	13.91	13.49
	12.25	1.94	3.60	19.29	18.73	18.18	17.62	17.06	16.50
	14.75	1.87	4.34	23.03	22.33	21.63	20.94	20.24	19.55
4	7.50	1.64	2.21	11.28	10.87	10.47	10.06	9.66	9.26
	10.50	1.52	3.09	15.37	14.76	14.15	13.54	12.93	12.32
3	5.50	1.23	1.63	7.42	7.02	6.63	6.23	5.83	5.43
	7.50	1.15	2.21	9.73	9.15	8.57	8.00	7.42	6.84

NOTE.—The safe loads given on bottom of p. 470 refer to two angles connected by plates so as to form a cross-section, as in accompanying figure. This form of strut is frequently used in light steel trusses.

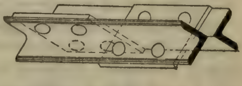
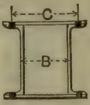


TABLE XVII.—SAFE LOADS IN TONS OF 2,000 LBS. FOR CHANNEL COLUMNS.



Allowed stresses per square inch;

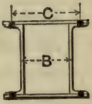
12,000 lbs. for lengths of 90 radii or under;

$17{,}100 - 57\frac{l}{r}$ for lengths over 90 radii, and less than 125 radii.

Size of plates, ins.	Weight of channels and plates.	r	Length in feet.					
			14	16	18	20	22	24
6″—8-lb. Channels. B = 3⅞″. C = 5¾″.								
Lattice. .	16.0	2.33	28.6	28.6	28.1	26.7	25.3	23.9
1/4×8	29.6	2.32	52.6	52.6	51.7	49.1	46.5	43.9
5/16×8	33.0	2.32	58.6	58.6	57.5	54.7	51.8	48.9
3/8×8	36.4	2.32	64.6	64.6	63.4	60.3	57.1	53.9
7/16×8	39.8	2.32	70.6	70.6	69.3	65.8	62.4	58.9
6″—15.5-lb. Channels. B = 3⅞″. C = 5¾″.								
Lattice. .	31.0	2.00	54.7	53.0	49.9	46.8		
5/16×8	48.0	2.12	84.7	84.2	79.7	75.1	70.5	
3/8×8	51.4	2.13	90.7	90.4	85.6	80.7	75.9	
7/16×8	54.8	2.14	96.7	96.7	91.5	86.4	81.2	
1/2×8	58.2	2.15	102.7	102.7	97.4	92.0	86.5	
7″—9.75-lb. Channels. B = 4⅝″. C = 6¾″.								
Lattice. .	19.5	2.72	34.2	34.2	34.2	34.2	32.8	31.4
1/4×9	34.8	2.67	61.2	61.2	61.2	61.2	58.5	55.9
5/16×9	38.6	2.67	67.9	67.9	67.9	67.9	65.0	62.0
3/8×9	42.5	2.66	74.7	74.7	74.7	74.4	71.2	68.0
7/16×9	46.3	2.66	81.5	81.5	81.5	81.1	77.6	74.1
7″—17.25-lb. Channels. B = 4⅝″. C = 6¾″.								
Lattice. .	34.5	2.43	60.8	60.8	60.8	58.1	55.3	52.4
5/16×9	53.6	2.49	94.6	94.6	94.6	91.4	87.1	82.8
3/8×9	57.5	2.50	101.3	101.3	101.3	98.2	93.5	88.9
7/16×9	61.3	2.50	108.1	108.1	108.1	104.8	99.9	95.0
1/2×9	65.1	2.51	114.8	114.8	114.8	111.5	106.3	101.1

To weight of channels and plates add the weight of rivets and lattice-bars. The size of lattice-bars should not be less than $1\frac{1}{2}\times\frac{5}{16}$ ins. for 6-inch channels, $1\frac{3}{4}\times\frac{5}{16}$ inch for 7- and 8-inch channels, or $2\times\frac{5}{16}$ ins. for 9- and 10-inch columns. See page 430.

TABLE XVII.—SAFE LOADS IN TONS OF 2,000 LBS. FOR CHANNEL COLUMNS (*continued*).



Allowed stresses per square inch:

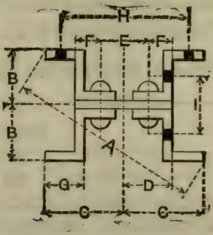
12,000 lbs. for lengths of 90 radii or under;

$17{,}100-57\frac{l}{r}$ for lengths over 90 radii, and less than 125 radii.

Size of plates, ins.	Weight of channels and plates.	r	Length in feet.					
			20	22	24	26	28	30
8″—11.25-lb. Channels. B = 5 7/16″. C = 7½″.								
Lattice. .	22.5	3.11	40.2	40.2	39.6	38.1	36.6	35.2
¼×10	39.5	3.03	70.2	70.2	68.4	65.7	63.1	60.5
5/16×10	43.7	3.02	77.7	77.7	75.5	72.6	69.7	66.7
⅜×10	48.0	3.01	85.2	85.2	82.7	79.4	76.2	73.0
8″—16.25-lb. Channels. B = 5 7/16″. C = 7½″.								
Lattice. .	32.5	2.89	57.4	56.8	54.6	52.3	50.1	47.8
⅜×10	58.0	2.92	102.4	101.9	97.9	93.9	89.9	85.9
7/16×10	62.3	2.91	109.9	109.3	105.0	100.7	96.4	92.1
½×10	66.5	2.91	117.4	116.7	112.1	107.5	102.9	98.3
9″—13.25-lb. Channels. B = 6 3/16″. C = 8½″.								
Lattice. .	26.5	3.49	46.7	46.7	46.7	46.7	45.2	43.6
¼×11	45.2	3.40	79.7	79.7	79.7	78.8	76.1	73.4
5/16×11	49.9	3.38	88.0	88.0	88.0	86.7	83.8	80.8
⅜×11	54.6	3.36	96.2	96.2	96.2	94.6	91.4	88.1
9″—20-lb. Channels. B = 6 3/16″. C = 8½″.								
Lattice. .	40.0	3.21	70.6	70.6	70.6	68.0	65.5	63.0
⅜×11	68.1	3.25	120.1	120.1	120.1	116.3	112.1	107.9
7/16×11	72.7	3.25	128.3	128.3	128.3	124.2	119.7	115.2
½×11	77.4	3.24	136.6	136.6	136.6	132.1	127.3	122.5
10″—15-lb. Channels. B = 7″. C = 9½″.								
Lattice. .	30.0	3.87	53.5	53.5	53.5	53.5	53.5	52.6
5/16×12	55.5	3.74	98.5	98.5	98.5	98.5	98.5	95.3
⅜×12	60.6	3.72	107.5	107.5	107.5	......	107.5	107.0
10″—25-lb. Channels. B = 7″. C = 9½″.								
Lattice. .	50.0	3.52	88.2	88.2	88.2	88.2	85.7	82.8
9/16×12	95.9	3.56	169.2	169.2	169.2	169.2	165.3	159.8
⅝×12	101.0	3.55	178.2	178.2	178.2	178.2	173.9	168.2

DIMENSIONS OF Z-BAR COLUMNS.

Carnegie Sections.



6″ columns.

4 Z-bars 3–3 1/16″ deep and 1 web-plate 5 3/4″ × thickness of Z-bars.

Diameter of bolt or rivet, 3/4″.	Thickness of metal.	A	B	C	D	E	F	G	H	I
	1/4	12 5/16	3 1/8	5 5/16	2 7/8	2 1/2	1 5/8	2 11/16	8 1/2	3 1/4
	5/16	12 3/8	3 7/32	5 5/16	2 7/8	2 1/2	1 5/8	2 3/4	8 3/8	3 3/8
	3/8	12 3/16	3 3/16	5 3/16	2 7/8	2 1/2	1 5/8	2 11/16	8 1/4	3 3/8
	7/16	12 1/4	3 9/32	5 3/16	2 7/8	2 1/2	1 5/8	2 3/4	8 1/8	3 1/2
	1/2	12	3 1/4	5 1/16	2 7/8	2 1/2	1 5/8	2 11/16	8	3 1/2
	9/16	12 1/16	3 11/32	5 1/16	2 7/8	2 1/2	1 5/8	2 3/4	7 7/8	3 5/8

8″ columns.

4 Z-bars 4–4 1/8″ deep and 1 web-plate 6 1/2″ × thickness of Z-bars.

Diameter of bolt or rivet, 3/4″.	Thickness of metal.	A	B	C	D	E	F	G	H	I
	1/4	14 11/16	4 1/8	6 1/16	3 1/4	3	1 3/4	3 1/16	9 1/2	4 1/4
	5/16	14 3/4	4 7/32	6 1/16	3 1/4	3	1 3/4	3 1/8	9 3/8	4 3/8
	3/8	14 13/16	4 5/16	6 1/16	3 1/4	3	1 3/4	3 3/16	9 1/4	4 1/2
	7/16	14 1/2	4 7/32	5 7/8	3 1/4	3	1 3/4	3 1/16	9 1/8	4 7/16
	1/2	14 9/16	4 5/16	5 7/8	3 1/4	3	1 3/4	3 1/8	9	4 9/16
	9/16	14 5/8	4 13/32	5 7/8	3 1/4	3	1 3/4	3 3/16	8 7/8	4 11/16
	5/8	14 1/4	4 5/16	5 11/16	3 1/4	3	1 3/4	3 1/16	8 3/4	4 5/8
	11/16	14 5/16	4 13/32	5 11/16	3 1/4	3	1 3/4	3 1/8	8 5/8	4 3/4
	3/4	14 3/8	4 1/2	5 11/16	3 1/4	3	1 3/4	3 3/16	8 1/2	4 7/8

TABLE XVIII.—SAFE LOADS IN TONS OF 2,000 LBS. FOR Z-BAR COLUMNS WITH SQUARE ENDS.

Allowed stresses per square inch $\begin{cases} 12{,}000 \text{ lbs. for lengths of 90 radii or under.} \\ 17{,}100 - 57\frac{l}{r} \text{ for lengths over 90 radii.} \end{cases}$

6″ Z-BAR COLUMNS.

Section: 4 Z-bars 3″ deep and one web-plate 5¾″ × thickness of Z-bars.

Length of column in feet.	¼ metal = 31.7 lbs. = 9.31 sq. in. r (min.) = 1.86.	5/16 metal = 39.8 lbs. = 11.7 sq. in. r (min.) = 1.90.	⅜ metal = 46.2 lbs. = 13.6 sq. in. r (min.) = 1.88.	7/16 metal = 54.3 lbs. = 16.0 sq. in. r (min.) = 1.93.	½ metal = 59.9 lbs. = 17.6 sq. in. r (min.) = 1.90.	9/16 metal = 67.9 lbs. = 20.0 sq. in. r (min.) = 1.95.
12 and under	55.9	70.3	81.6	95.8	105.7	119.8
14	55.7	70.3	81.6	95.8	105.7	119.8
16	52.3	66.5	76.6	91.3	99.9	114.8
18	48.8	62.3	71.7	85.6	93.6	107.8
20	45.4	58.1	66.7	79.9	87.2	100.8
22	42.0	53.9	61.8	74.3	80.9	93.8
24	38.6	49.7	56.9	68.6	74.6	86.8
26	35.2	45.5	51.9	63.0	68.2	79.8
28	31.7	41.3	47.0	57.3	61.9	72.8
30	28.3	37.1	42.0	51.7	55.5	65.8

8″ Z-BAR COLUMNS.

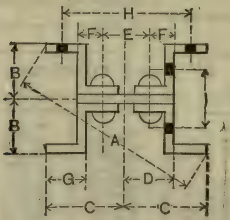
Section: 4 Z-bars 4″ deep and 1 web-plate 6½″ × thickness of Z-bars.

Length of column in feet.	¼ metal = 38.3 lbs. = 11.3 sq. in. r (min.) = 2.47.	5/16 metal = 48.1 lbs. = 14.1 sq. in. r (min.) = 2.52.	⅜ metal = 58.0 lbs. = 17.1 sq. in. r (min.) = 2.57.	7/16 metal = 64.7 lbs. = 19.0 sq. in. r (min.) = 2.49.	½ metal = 74.4 lbs. = 21.9 sq. in. r (min.) = 2.55.	9/16 metal = 84.1 lbs. = 24.8 sq. in. r (min.) = 2.60.	⅝ metal = 89.2 lbs. = 26.3 sq. in. r (min.) = 2.52.	11/16 metal = 98.8 lbs. = 29.0 sq. in. r (min.) = 2.58.	¾ metal = 108.4 lbs. = 31.9 sq. in. r (min.) = 2.63.
18 and under	67.5	84.8	102.4	114.2	131.2	148.5	157.5	174.3	191.2
20	65.0	82.5	100.5	110.5	128.2	146.4	153.3	171.3	189.6
22	61.9	78.7	95.9	105.3	122.4	139.9	146.2	163.5	181.3
24	58.8	74.8	91.3	100.1	116.5	133.4	139.1	155.8	173.0
26	55.7	71.0	86.8	94.8	110.6	126.9	132.0	148.1	164.7
28	52.6	67.1	82.3	89.6	104.7	120.3	124.8	140.4	156.4
30	49.4	63.3	77.7	84.4	98.8	113.8	117.7	132.7	148.2
32	46.3	59.5	73.2	79.2	93.0	107.3	110.6	125.0	139.9
34	43.2	55.6	68.7	74.0	87.1	100.8	103.5	117.3	131.6
36	40.1	51.8	64.1	68.7	81.2	94.3	96.4	109.6	123.3
38	37.0	48.0	59.6	63.5	75.3	87.8	89.4	101.9	115.0
40	33.9	44.1	55.0	58.3	69.5	81.3	82.2	94.2	106.7

To the above weights of column shafts add the weight of rivets.

DIMENSIONS OF Z-BAR COLUMNS.

Carnegie Sections.



10″ columns.

4 Z-bars 5–5 1/8″ deep and one web-plate 7″ × thickness of Z-bars.

	Thickness of metal.	A	B	C	D	E	F	G	H	I
Diameter of bolt or rivet, 3/4″.	5/16	16 1/2	5 5/32	6 7/16	3 1/2	3 1/4	1 7/8	3 1/4	10 1/8	5 5/16
	3/8	16 9/16	5 1/4	6 7/16	3 1/2	3 1/4	1 7/8	3 5/16	10	5 7/16
	7/16	16 5/8	5 11/32	6 7/16	3 1/2	3 1/4	1 7/8	3 3/8	9 7/8	5 9/16
	1/2	16 3/8	5 1/4	6 1/4	3 1/2	3 1/4	1 7/8	3 1/4	9 3/4	5 1/2
	9/16	16 7/16	5 11/32	6 1/4	3 1/2	3 1/4	1 7/8	3 5/16	9 5/8	5 5/8
	5/8	16 1/2	5 7/16	6 1/4	3 1/2	3 1/4	1 7/8	3 3/8	9 1/2	5 3/4
	11/16	16 3/16	5 11/32	6 1/16	3 1/2	3 1/4	1 7/8	3 1/4	9 3/8	5 11/16
	3/4	16 1/4	5 7/16	6 1/16	3 1/2	3 1/4	1 7/8	3 5/16	9 1/4	5 13/16
	13/16	16 5/16	5 17/32	6 1/16	3 1/2	3 1/4	1 7/8	3 3/8	9 1/8	5 15/16

12″ columns.

4 Z-bars 6–6 1/8″ deep and one web-plate 8″ × thickness of Z-bars.

	Thickness of metal.	A	B	C	D	E	F	G	H	I
Diameter of bolt or rivet, 3/4″.	3/8	18 7/8	6 3/16	7 1/8	4	4	2	3 1/2	11 1/4	6 3/8
	7/16	18 15/16	6 9/32	7 1/8	4	4	2	3 9/16	11 1/8	6 1/2
	1/2	19	6 3/8	7 1/8	4	4	2	3 5/8	11	6 5/8
	9/16	18 11/16	6 9/32	6 15/16	4	4	2	3 1/2	10 7/8	6 9/16
	5/8	18 3/4	6 3/8	6 15/16	4	4	2	3 9/16	10 3/4	6 11/16
	11/16	18 13/16	6 15/32	6 15/16	4	4	2	3 5/8	10 5/8	6 13/16
	3/4	18 9/16	6 3/8	6 3/4	4	4	2	3 1/2	10 1/2	6 3/4
	13/16	18 5/8	6 15/32	6 3/4	4	4	2	3 9/16	10 3/8	6 7/8
	7/8	18 11/16	6 9/16	6 3/4	4	4	2	3 5/8	10 1/4	7

TABLE XVIII.—SAFE LOADS IN TONS OF 2,000 LBS. FOR Z-BAR COLUMNS WITH SQUARE ENDS (*continued*).

Allowed stresses per square inch $\begin{cases} 12{,}000 \text{ lbs. for lengths of 90 radii or under.} \\ 17{,}100 - 57\dfrac{l}{r} \text{ for lengths of over 90 radii.} \end{cases}$

10″ Z-BAR COLUMNS.

Section: 4 Z-bars 5″ deep and one web-plate 7″ × thickness of Z-bars.

Length of column in feet.	5/16 metal = 53.7 lbs. = 15.8 sq. in. r (min.) = 3.08.	3/8 metal = 64.7 lbs. = 19.0 sq. in. r (min.) = 3.13.	7/16 metal = 75.8 lbs. = 22.3 sq. in. r (min.) = 3.18.	1/2 metal = 83.3 lbs. = 24.5 sq. in. r (min.) = 3.10.	9/16 metal = 94.2 lbs. = 27.7 sq. in. r (min.) = 3.15.	5/8 metal = 105.2 lbs. = 30.9 sq. in. r (min.) = 3.21.	11/16 metal = 111.0 lbs. = 32.7 sq. in. r (min.) = 3.13.	3/4 metal = 121.8 lbs. = 35.8 sq. in. r (min.) = 3.18.	13/16 metal = 132.6 lbs. = 39.0 sq. in. r (min.) = 3.25.
22 and under	94.7	114.2	133.9	147.0	166.2	185.6	196.0	214.9	234.0
24	92.8	112.6	133.1	144.6	164.8	185.3	193.6	213.9	234.0
26	89.3	108.6	128.3	139.2	158.7	178.7	186.5	206.2	226.6
28	85.8	104.4	123.5	133.8	152.7	172.1	179.3	198.5	218.4
30	82.3	100.2	118.7	128.4	146.7	165.5	172.2	190.8	210.2
32	78.8	96.1	113.8	123.0	140.7	158.9	165.0	183.1	202.0
34	75.3	91.9	109.1	117.6	134.7	152.3	157.9	175.4	193.8
36	71.8	87.8	104.3	112.2	128.7	145.7	150.7	167.8	185.6
38	68.3	83.6	99.5	106.8	122.7	139.1	143.6	160.0	177.4
40	64.8	79.4	94.7	101.4	116.7	132.5	136.5	152.3	169.1
42	61.3	75.3	89.9	96.0	110.6	125.9	129.4	144.6	160.9
44	57.7	71.1	85.1	90.6	104.6	119.3	122.2	136.9	152.7
46	54.2	67.0	80.3	85.2	98.6	112.7	115.1	129.2	144.5
48	50.7	62.8	75.5	79.8	92.6	106.1	107.9	121.5	136.3
50	47.2	58.6	70.7	74.4	86.6	99.5	100.8	113.8	128.1

12″ Z-BAR COLUMNS.

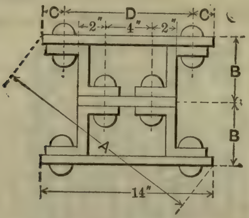
Section: 4 Z-bars 6″ deep and one web-plate 8″ × thickness of Z-bars.

Length of column in feet.	3/8 metal = 72.7 lbs. = 21.4 sq. in. r (min.) = 3.67.	7/16 metal = 85.2 lbs. = 25.0 sq. in. r (min.) = 3.72.	1/2 metal = 97.8 lbs. = 28.8 sq. in. r (min.) = 3.77.	9/16 metal = 106.2 lbs. = 31.2 sq. in. r (min.) = 3.70.	5/8 metal = 118.5 lbs. = 34.8 sq. in. r (min.) = 3.75.	11/16 metal = 130.9 lbs. = 38.5 sq. in. r (min.) = 3.73.	3/4 metal = 137.8 lbs. = 40.5 sq. in. r (min.) = 3.68.	13/16 metal = 149.9 lbs. = 44.1 sq. in. r (min.) = 3.66.	7/8 metal = 162.1 lbs. = 47.7 sq. in. r (min.) = 3.64.
26 and under	128.3	150.3	172.6	187.3	209.1	231.0	243.0	264.5	286.1
28	127.0	149.7	172.5	186.0	208.9	230.3	240.8	261.4	282.1
30	123.0	145.1	167.6	180.2	202.5	223.3	233.2	253.2	273.2
32	119.0	140.5	162.4	174.5	196.1	216.3	225.7	245.0	264.2
34	115.1	135.9	157.2	168.7	189.8	209.2	218.2	236.7	255.2
36	111.1	131.3	152.0	162.9	183.4	202.1	210.6	228.4	246.3
38	107.1	126.7	146.8	157.1	177.0	195.1	203.1	220.2	237.3
40	103.1	122.1	141.5	151.4	170.7	188.0	195.6	211.9	228.3
42	99.1	117.5	136.3	145.5	164.4	180.9	188.0	203.7	219.4
44	95.1	112.9	131.1	139.8	158.0	173.9	180.5	195.5	210.4
46	91.2	108.3	126.2	134.0	151.6	166.8	172.9	187.2	201.4
48	87.2	103.6	120.7	128.2	145.3	159.8	165.4	179.0	192.4
50	83.2	99.1	115.5	122.4	138.9	152.7	157.9	170.7	183.5

To the above weights of column shafts add the weight of rivets.

DIMENSIONS OF Z-BAR COLUMNS.

Carnegie Sections.



14″ columns.

Section: 4 Z-bars $6\frac{1}{8}'' \times \frac{11}{16}''$. 1 web-plate $8'' \times \frac{11}{16}''$. 2 side plates 14″ wide.

Diameter of bolt or rivet, $\frac{7}{8}''$.	Thickness of side plates.	A	B	C	D
	3/8	$19\frac{9}{16}$	$6\frac{27}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	7/16	$19\frac{11}{16}$	$6\frac{29}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	1/2	$19\frac{3}{4}$	$6\frac{31}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	9/16	$19\frac{7}{8}$	$7\frac{1}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	5/8	$19\frac{15}{16}$	$7\frac{3}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	11/16	$20\frac{1}{16}$	$7\frac{5}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	3/4	$20\frac{1}{8}$	$7\frac{7}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	13/16	$20\frac{1}{4}$	$7\frac{9}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$
	7/8	$20\frac{5}{16}$	$7\frac{11}{32}$	$1\frac{11}{16}$	$10\frac{5}{8}$

14″ columns.

Section: 4 Z-bars $6'' \times \frac{3}{4}''$. 1 web-plate $8'' \times \frac{3}{4}''$. 2 side plates 14″ wide.

Diameter of bolt or rivet, $\frac{7}{8}''$.	Thickness of side plates.	A	B	C	D
	3/8	$19\frac{7}{16}$	$6\frac{3}{4}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	7/16	$19\frac{1}{2}$	$6\frac{13}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	1/2	$19\frac{5}{8}$	$6\frac{7}{8}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	9/16	$19\frac{3}{4}$	$6\frac{15}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	5/8	$19\frac{13}{16}$	7	$1\frac{3}{4}$	$10\frac{1}{2}$
	11/16	$19\frac{7}{8}$	$7\frac{1}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	3/4	20	$7\frac{1}{8}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	13/16	$20\frac{1}{16}$	$7\frac{3}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	7/8	$20\frac{1}{8}$	$7\frac{1}{4}$	$1\frac{3}{4}$	$10\frac{1}{2}$

TABLE XVIII.—SAFE LOADS IN TONS OF 2,000 LBS. FOR STEEL Z-BAR COLUMNS, WITH SQUARE ENDS (*continued*).

Allowed strains per square inch $\begin{cases} \text{12,000 lbs. for lengths of 90 radii or under.} \\ 17{,}100 - 57\,\dfrac{l}{r} \text{ for lengths over 90 radii.} \end{cases}$

14″ Z-BAR COLUMNS.

Section: 4 Z-bars $6\frac{1}{8}'' \times \frac{11}{16}''$. 1 web-plate $8'' \times \frac{11}{16}''$. 2 side plates 14″ wide.

Length of column in feet.	14×3/8 plates = 166.6 lbs. = 49.0 sq. in. r (min.) = 3.80.	14×7/16 plates = 172.6 lbs. = 50.8 sq. in. r (min.) = 3.81.	14×1/2 plates = 178.5 lbs. = 52.5 sq. in. r (min.) = 3.82.	14×9/16 plates = 184.5 lbs. = 54.3 sq. in. r (min.) = 3.82.	14×5/8 plates = 190.4 lbs. = 56.0 sq. in. r (min.) = 3.83.	14×11/16 plates = 196.4 lbs. = 57.8 sq. in. r (min.) = 3.84.	14×3/4 plates = 202.3 lbs. = 59.5 sq. in. r (min.) = 3.85.	14×13/16 plates = 208.4 lbs. = 61.3 sq. in. r (min.) = 3.85.	14×7/8 plates = 214.2 lbs. = 63.0 sq. in. r (min.) = 3.85.
28 and under	294.0	304.5	315.0	325.5	336.0	346.5	357.0	367.5	378.0
30	286.6	297.2	307.7	318.3	328.9	339.5	350.0	360.4	370.9
32	277.8	288.1	298.3	308.6	318.9	329.2	339.4	349.5	359.7
34	269.0	278.9	288.9	298.9	308.9	318.9	328.8	338.6	348.6
36	260.1	269.8	279.5	289.2	298.9	308.6	318.2	327.7	337.4
38	251.3	260.7	270.1	279.5	289.0	298.3	307.6	316.8	326.2
40	242.5	251.6	260.7	269.7	278.9	288.0	297.0	306.0	315.0
42	233.7	242.5	251.3	260.1	269.0	277.8	286.4	295.1	303.8
44	224.9	233.3	241.9	250.4	258.9	267.4	275.8	284.2	292.6
46	216.0	224.3	232.4	240.7	249.0	257.2	265.2	273.3	281.5
48	207.2	215.1	223.0	230.9	238.9	246.9	254.6	262.4	270.3
50	198.4	206.0	213.6	221.3	229.0	236.5	244.0	251.5	259.1

14″ Z-BAR COLUMNS.

Section: 4 Z-bars $6'' \times \frac{3}{4}''$. 1 web-plate $8'' \times \frac{3}{4}''$. 2 side plates 14″ wide.

Length of column in feet.	14×3/8 plates = 173.4 lbs. = 51.0 sq. in. r (min.) = 3.75.	14×7/16 plates = 179.4 lbs. = 52.8 sq. in. r (min.) = 3.76.	14×1/2 plates = 185.3 lbs. = 54.5 sq. in. r (min.) = 3.77.	14×9/16 plates = 191.3 lbs. = 56.3 sq. in. r (min.) = 3.78.	14×5/8 plates = 197.2 lbs. = 58.0 sq. in. r (min.) = 3.79.	14×11/16 plates = 203.2 lbs. = 59.8 sq. in. r (min.) = 3.80.	14×3/4 plates = 209.1 lbs. = 61.5 sq. in. r (min.) = 3.80.	14×13/16 plates = 215.1 lbs. = 63.3 sq. in. r (min.) = 3.81.	14×7/8 plates = 221.0 lbs. = 65.0 sq. in. r (min.) = 3.82.
28 and under	306.0	316.5	327.0	337.5	348.0	358.5	369.0	379.5	390.0
30	296.7	307.2	317.8	328.3	338.9	349.4	359.9	370.5	381.1
32	287.4	297.6	307.9	318.2	328.4	338.7	348.9	359.1	369.4
34	278.1	288.0	298.0	308.0	318.0	327.9	337.8	347.8	357.8
36	268.8	278.4	288.2	297.9	307.4	317.2	326.8	336.4	346.1
38	259.5	268.8	278.3	287.7	297.0	306.4	315.7	325.1	334.5
40	250.2	259.3	268.4	277.5	286.5	295.6	304.7	313.7	322.8
42	240.9	249.7	258.5	267.3	276.1	284.8	293.6	302.4	311.2
44	231.6	240.1	248.6	257.1	265.6	274.1	282.5	291.0	299.6
46	222.4	230.5	238.7	246.9	255.1	263.4	271.5	279.7	287.9
48	213.0	220.9	228.8	236.8	244.7	252.6	260.4	268.3	276.2
50	203.7	211.3	219.0	226.6	234.2	241.8	249.4	257.0	264.6

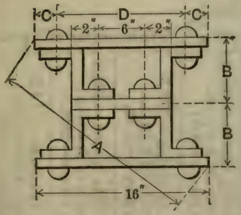
To the above weight of column shafts add the weight of rivets.

DIMENSIONS OF Z-BAR COLUMNS.

CARNEGIE SECTIONS.

16″ COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web-plate 10″ × 1″. 2 side plates 16″ wide.

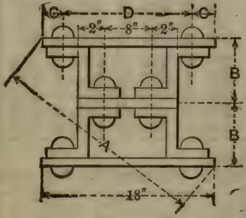


Diam. of bolts or rivets, ⅞″.

Thickness of side plates.	A	B	C	D
1/2	21 7/16	7 1/8	1 7/8	12 1/4
9/16	21 1/2	7 3/16	1 7/8	12 1/4
5/8	21 9/16	7 1/4	1 7/8	12 1/4
11/16	21 11/16	7 5/16	1 7/8	12 1/4
3/4	21 3/4	7 3/8	1 7/8	12 1/4
13/16	21 13/16	7 7/16	1 7/8	12 1/4
7/8	21 15/16	7 1/2	1 7/8	12 1/4
15/16	22	7 9/16	1 7/8	12 1/4
1	22 1/16	7 5/8	1 7/8	12 1/4

18″ COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web-plate 12″ × 1″.
2 side plates 18″ wide.



Diam. of bolts or rivets, ⅞″.

Thickness of side plates.	A	B	C	D
1/2	23	7 1/8	1 7/8	14 1/4
9/16	23 1/16	7 3/16	1 7/8	14 1/4
5/8	23 1/8	7 1/4	1 7/8	14 1/4
11/16	23 3/16	7 5/16	1 7/8	14 1/4
3/4	23 1/4	7 3/8	1 7/8	14 1/4
13/16	23 5/16	7 7/16	1 7/8	14 1/4
7/8	23 7/16	7 1/2	1 7/8	14 1/4
15/16	23 1/2	7 9/16	1 7/8	14 1/4
1	23 5/8	7 5/8	1 7/8	14 1/4

TABLE XVIII.—SAFE LOADS IN TONS OF 2,000 LBS. FOR STEEL Z-BAR COLUMNS WITH SQUARE ENDS (*continued*).

Allowed strains per square inch: { 12,000 lbs. for lengths of 90 radii or under. $17{,}100 - 57\frac{l}{r}$ for lengths over 90 radii.

16″ Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web-plate 10″ × 1″. 2 side plates 16″ wide.

Length of column in feet.	16 × ½ plates = 226.7 lbs. = 66.7 sq. in. r (min.) = 4.50.	16 × 9/16 plates = 233.5 lbs. = 68.7 sq. in. r (min.) = 4.50.	16 × ⅝ plates = 240.3 lbs. = 70.7 sq. in. r (min.) = 4.50.	16 × 11/16 plates = 247.1 lbs. = 72.7 sq. in. r (min.) = 4.51.	16 × ¾ plates = 253.9 lbs. = 74.7 sq. in. r (min.) = 4.51.	16 × 13/16 plates = 260.7 lbs. = 76.7 sq. in. r (min.) = 4.51.	16 × ⅞ plates = 267.5 lbs. = 78.7 sq. in. r (min.) = 4.52.	16 × 15/16 plates = 274.3 lbs. = 80.7 sq. in. r (min.) = 4.52.	16 × 1 plates = 281.1 lbs. = 82.7 sq. in. r (min.) = 4.52.
32 and under	400.1	412.1	424.1	436.1	448.1	460.1	472.1	484.1	496.1
34	397.7	409.8	421.9	433.9	446.0	458.1	470.2	482.2	494.2
36	387.6	399.3	411.1	422.9	434.7	446.5	458.2	470.0	481.8
38	377.5	388.9	400.4	411.8	423.4	434.8	446.3	457.9	469.3
40	367.3	378.5	389.6	400.9	412.1	423.2	434.4	445.6	456.7
42	357.1	368.0	378.9	389.8	400.7	411.6	422.5	433.4	444.2
44	347.0	357.6	368.2	378.8	389.4	400.0	410.5	421.1	431.7
46	336.9	347.1	357.4	367.7	378.1	388.4	398.6	409.0	419.2
48	326.7	336.7	346.7	356.7	366.7	376.8	386.7	396.7	406.7
50	316.6	326.3	336.0	345.7	355.4	365.1	374.8	384.5	394.2

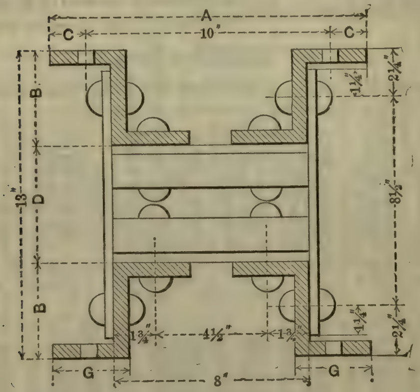
18″ Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web-plate 12″ × 1″. 2 side plates 18″ wide.

Length of column in feet.	18 × ½ plates = 240.4 lbs. = 70.7 sq. in. r (min.) = 4.81.	18 × 9/16 plates = 248.0 lbs. = 72.9 sq. in. r (min.) = 4.81.	18 × ⅝ plates = 255.7 lbs. = 75.2 sq. in. r (min.) = 4.90.	18 × 11/16 plates = 263.0 lbs. = 77.4 sq. in. r (min.) = 4.98.	18 × ¾ plates = 271.0 lbs. = 79.7 sq. in. r (min.) = 5.06.	18 × 13/16 plates = 278.6 lbs. = 81.9 sq. in. r (min.) = 5.14.	18 × ⅞ plates = 286.3 lbs. = 84.2 sq. in. r (min.) = 5.22.	18 × 15/16 plates = 293.9 lbs. = 86.4 sq. in. r (min.) = 5.26.	18 × 1 plates = 301.6 lbs. = 88.7 sq. in. r (min.) = 5.26.
34 and under	424.1	437.6	451.1	464.6	478.1	491.6	505.1	518.6	532.1
36	419.7	436.8	451.1	464.6	478.1	491.6	505.1	518.6	532.1
38	409.4	426.4	443.2	456.2	476.8	491.6	505.1	518.6	532.1
40	399.2	416.0	432.7	449.5	466.0	482.6	499.1	514.2	527.5
42	388.9	405.6	422.3	438.8	455.3	471.7	488.1	503.0	516.0
44	378.7	395.2	411.7	428.2	444.5	460.8	477.0	491.8	504.5
46	368.4	384.9	401.2	417.5	433.8	449.9	466.0	480.5	493.0
48	358.1	374.5	390.7	406.9	423.0	439.0	454.9	469.3	481.4
50	347.9	364.1	380.2	396.2	412.2	428.1	443.9	458.1	469.9

DIMENSIONS OF CONSTANT-DIMENSION Z-BAR COLUMNS.

(CARNEGIE SECTIONS.)



Constant dimensions are given on the sketch above for all columns. Variable dimensions, see below. All rivets ¾″ diameter. Open holes for ¾″ rivets or bolts.

Web tie-plates 9″ × 5/16″ × 0′—8″, Flange tie-plates 9″ × 5/16″ × 0′—11″ } for all columns less than ⅝″ metal.

For all columns ⅝″ metal and over, tie-plates are ⅜″ thick.

All tie-plates spaced about 3′-0″ centre to centre.

Size of Z-bars.	A	B	C	D	G
4″ × 3 1/16″ × ¼″	13⅝″	4″	1 13/16″	5″	3 1/16″
4 1/16″ × 3⅛″ × 5/16″	13⅝″	4 1/16″	1 13/16″	4⅞″	3⅛″
4⅛″ × 3 3/16″ × ⅜″	13⅝″	4⅛″	1 13/16″	4¾″	3 3/16″
4″ × 3 1/16″ × 7/16″	13¼″	4″	1⅝″	5″	3 1/16″
4 1/16″ × 3⅛″ × ½″	13¼″	4 1/16″	1⅝″	4⅞″	3⅛″
4⅛″ × 3 3/16″ × 9/16″	13¼″	4⅛″	1⅝″	4¾″	3 3/16″
4″ × 3 1/16″ × ⅝″	12⅞″	4″	1 7/16″	5″	3 1/16″
4 1/16″ × 3⅛″ × 11/16″	12⅞″	4 1/16″	1 7/16″	4⅞″	3⅛″
4⅛″ × 3 3/16″ × ¾″	12⅞″	4⅛″	1 7/16″	4¾″	3 3/16″

TABLE XIX.—SAFE LOADS IN TONS OF 2,000 LBS. FOR CONSTANT-DIMENSION Z-BAR COLUMNS, SQUARE ENDS.

Allowed stresses per square inch: 12,000 lbs. for lengths of 90 radii or under; $17{,}100 - 57\frac{l}{r}$ for lengths over 90 radii.

Section: 4 Z-bars 4″ deep with tie-plates.

Length of column in feet.	4 Z's 4″ × 3 1/16″ × 1/4″ 9.64 sq. in. 32.8 lbs. r (min.) 4.0952.	4 Z's 4 1/16 × 3 1/8 × 5/16″ 12.12 sq. in. 41.2 lbs. r (min.) 4.0714.	4 Z's 4 1/8 × 3 3/16 × 3/8″ 14.64 sq. in. 49.6 lbs. r (min.) 4.0478.	4 Z's 4 × 3 1/16 × 7/16″ 16.20 sq. in. 55.2 lbs. r (min.) 3.9949.	4 Z's 4 1/16 × 3 1/8 × 1/2″ 18.64 sq. in. 63.2 lbs. r (min.) 3.9712.	4 Z's 4 1/8 × 3 3/16 × 9/16″ 21.08 sq. in. 71.6 lbs. r (min.) 3.9479.	4 Z's 4″ × 3 1/16″ × 5/8″ 22.20 sq. in. 75.6 lbs. r (min.) 3.8949.	4 Z's 4 1/16 × 3 1/8 × 11/16″ 24.56 sq. in. 83.6 lbs. r (min.) 3.8715.	4 Z's 4 1/8 × 3 3/16 × 3/4″ 27.00 sq. in. 91.6 lbs. r (min.) 3.8478.
28 and under	......	......	......	97.2	111.8	126.5	133.2	147.4	162.0
30	57.8	72.7	87.8	96.9	111.2	125.5	131.3	144.9	158.9
32	56.7	71.1	85.6	94.1	108.0	121.8	127.4	140.6	154.1
34	55.0	69.0	83.1	91.4	104.8	118.2	123.5	136.2	149.3
36	53.4	67.0	80.6	88.6	101.6	114.5	119.6	131.9	144.5
40	50.2	62.9	75.7	83.0	95.2	107.2	111.8	123.2	134.9

Section: 4 Z-bars 4″ deep; 4″ × 3 1/16″ × 5/8″ with web-plates.

Length of column in feet.	2 plates 8″ × 3/8″ 28.20 sq. in. 96 lbs. r (min.) 3.6163.	2 plates 8″ × 7/16″ 29.20 sq. in. 99.4 lbs. r (min.) 3.5794.	2 plates 8″ × 1/2″ 30.20 sq. in. 103.8 lbs. r (min.) 3.5447.	2 plates 8″ × 9/16″ 31.20 sq. in. 106.2 lbs. r (min.) 3.5118.	2 plates 8″ × 5/8″ 32.20 sq. in. 109.6 lbs. r (min.) 3.4807.	2 plates 8″ × 3/4″ 34.2 sq. in. 116.3 lbs. r (min.) 3.4233.	2 plates 8″ × 7/8″ 36.2 sq. in. 123.1 lbs. r (min.) 3.3714.	2 plates 8″ × 1″ 38.2 sq. in. 129.9 lbs. r (min.) 3.3242.	2 plates 8″ × 1 1/8″ 40.2 sq. in. 136.7 lbs. r (min.) 3.2811.
26 and under	169.2	175.2	181.2	187.2	193.2	205.2	217.2	229.2	241.2
28	166.4	171.5	176.6	181.7	186.7	196.7	206.7	216.6	226.4
30	161.1	166.0	170.8	175.6	180.4	189.9	199.3	208.7	218.0
32	155.8	160.4	165.0	169.5	174.1	183.1	192.0	200.9	209.6
34	150.4	154.8	159.1	163.5	167.7	176.2	184.7	193.0	201.3
36	145.1	149.2	153.3	157.4	161.4	169.4	177.3	185.1	192.9
40	134.4	138.1	141.7	145.2	148.8	155.7	162.6	169.4	176.1

Section: 4 Z-bars 4″ deep; 4 1/8″ × 3 3/16″ × 3/4″ with web-plates.

Length of column in feet.	2 plates 8″ × 7/8″ 41 sq. in. 139.4 lbs. r (min.) 3.4017.	2 plates 8″ × 1″ 43 sq. in. 146.2 lbs. r (min.) 3.3587.	2 plates 8″ × 1 1/8″ 45 sq. in. 153 lbs. r (min.) 3.3192.	2 plates 8″ × 1 1/4″ 47 sq. in. 159.8 lbs. r (min.) 3.2825.	2 plates 8″ × 1 3/8″ 49 sq. in. 166.6 lbs. r (min.) 3.2485.	2 plates 8″ × 1 1/2″ 51 sq. in. 173.4 lbs. r (min.) 3.2169.	2 plates 8″ × 1 5/8″ 53 sq. in. 180.2 lbs. r (min.) 3.1873.	2 plates 8″ × 1 3/4″ 55 sq. in. 187 lbs. r (min.) 3.1597.	2 plates 8″ × 1 7/8″ 57 sq. in. 193.8 lbs. r (min.) 3.1337.
22 and under	......	......	......	......	......	......	318.0	330.0	342.0
24	246.0	258.0	270.0	282.0	294.0	306.0	316.7	327.4	338.1
26	243.4	253.8	264.2	274.5	284.8	295.1	305.3	315.5	325.6
28	235.1	245.1	254.9	264.7	274.5	284.2	293.9	303.6	313.2
30	226.9	236.3	245.7	254.9	264.2	273.4	282.6	291.7	300.7
32	218.7	227.5	236.4	245.2	253.9	262.6	271.2	279.8	288.3
34	210.4	218.8	227.1	235.4	243.6	251.7	259.8	267.8	275.9
36	202.2	210.0	217.8	225.6	233.2	240.9	248.4	255.9	263.4
40	185.7	192.5	199.3	206.0	212.6	219.2	225.7	232.1	238.5

TABLE XX.—DIMENSIONS AND SAFE CONCENTRIC LOADS FOR PHŒNIX STEEL COLUMNS.

(*For description of column, see page* 440.)

The dimensions given in the following table are subject to such slight variations as are unavoidable in the manufacture of these shapes.

The weights given are those of the segments composing the columns, and from 2 to 5 per cent. must be added for weight of the rivet-heads.

The A, B¹, B², and C columns have each 4 segments, the E have 6, and the G have 8 segments.

Any desired thickness between the minimum and maximum can be furnished.

Allowed strains per sq. inch: 14,000 lbs. for lengths of 70 radii or under; $17{,}100 - 57\frac{l}{r}$, for lengths over 70 radii.

One segment.		Diameters in inches.			One column.			Safe load in net tons for	
Thickness in inches.	Weight in lbs. per yard.	d Inside.	D Outside.	D^1 Over flanges.	Area of cross-section, square inch.	Weight per foot, in pounds.	Least radius of gyration, in inches.	12-foot lengths.	16-foot lengths.
3/16	9.7	**A**	4	6 1/16	3.8	12.9	1.45	21.7	18.1
1/4	12.2	3 5/8	4 1/8	6 3/16	4.8	16.3	1.50	27.9	23.9
5/16	14.8		4 1/4	6 5/16	5.8	19.7	1.55	34.2	29.0
3/8	17.3		4 3/8	6 7/16	6.8	23.1	1.59	40.5	34.7
1/4	16.3		5 3/8	8 1/8	6.4	21.8	1.95	41.2	36.6
5/16	19.9		5 1/2	8 3/16	7.8	26.5	2.00	50.7	45.3
3/8	23.5	**B.1**	5 5/8	8 5/16	9.2	31.3	2.04	59.8	54.0
7/16	27.0	4 7/8	5 3/4	8 7/16	10.6	36.0	2.09	69.5	62.8
1/2	30.6		5 7/8	8 1/2	12.0	40.8	2.13	84.0	71.7
9/16	34.2		6	8 9/16	13.4	45.6	2.18	93.8	80.9
5/8	37.7		6 1/8	8 11/16	14.8	50.3	2.23	103.6	90.2
1/4	18.9		6 9/16	9 1/4	7.4	25.2	2.39	51.8	46.3
5/16	22.9		6 11/16	9 3/8	9.0	30.6	2.43	63.0	56.7
3/8	27.0	**B.2**	6 13/16	9 7/16	10.6	36.0	2.48	74.2	67.2
7/16	31.1	6 1/16	6 15/16	9 1/2	12.2	41.5	2.52	85.4	77.8
1/2	35.2		7 1/16	9 5/8	13.8	46.9	2.57	96.6	88.6
9/16	39.3		7 3/16	9 3/4	15.4	52.4	2.61	107.8	99.4
5/8	43.3		7 5/16	9 13/16	17.0	57.8	2.66	119.0	110.4

Least radius of gyration equals $D \times .3636$.

TABLE XX.—SAFE LOAD IN TONS OF 2,000 LBS. PHŒNIX STEEL COLUMNS (*continued*).

ALLOWED STRAINS PER SQUARE INCH, 14,000 LBS.

One segment.		Diameters in inches.			One column.			Safe load for 16-foot lengths and under.
Thickness in inches.	Weight in lbs. per yard.	*d* Inside.	*D* Outside.	D^1 Over flanges.	Area of cross-section, sq. in.	Weight per foot in pounds.	Least radius of gyration, in inches.	
¼	25½		7 13/16	11 11/16	10.0	34.0	2.84	70.0
5/16	31		7 15/16	11¾	12.1	41.3	2.88	84.7
⅜	36		8 1/16	11 13/16	14.1	48.0	2.93	98.7
7/16	41		8 3/16	11⅞	16.0	54.6	2.97	112.0
½	46		8 5/16	11 15/16	18.0	61.3	3.01	126.0
9/16	51	**C**	8 7/16	12	19.9	68.0	3.06	139.3
⅝	56		8 9/16	12 1/16	21.9	74.6	3.11	153.3
11/16	62	7⅜	8 11/16	12 3/16	24.3	82.6	3.16	170.1
¾	68		8 13/16	12 5/16	26.6	90.6	3.20	186.2
13/16	73		8 15/16	12 7/16	28.6	97.3	3.24	200.3
⅞	78		9 1/16	12½	30.6	104.0	3.29	214.2
1	89		9 5/16	12⅝	34.8	118.6	3.34	243.6
1⅛	99		9 9/16	12 13/16	38.8	132.0	3.48	271.7
1¼	109		9 11/16	13	42.7	145.3	3.57	298.9
¼	28		11 9/16	15½	16.5	56.0	4.20	115.3
5/16	32½		11 11/16	15⅝	19.1	65.0	4.25	133.8
⅜	37		11 13/16	15¾	21.7	74.0	4.29	152.0
7/16	42		11 15/16	15⅞	24.7	84.0	4.34	173.0
½	47		12 1/16	15 15/16	27.6	94.0	4.38	193.2
9/16	52		12 3/16	16 1/16	30.6	104.0	4.43	214.1
⅝	57	**E**	12 5/16	16 3/16	33.5	114.0	4.48	234.7
11/16	62	11 1/16	12 7/16	16 5/16	36.4	124.0	4.52	255.0
¾	68		12 9/16	16 7/16	40.0	136.0	4.56	280.0
13/16	73		12 11/16	16 9/16	43.0	146.0	4.61	300.6
⅞	78		12 13/16	16 11/16	45.9	156.0	4.66	321.2
1	88		13 1/16	16 13/16	51.7	176.0	4.73	362.0
1⅛	98		13 5/16	17 1/16	57.6	196.0	4.84	403.2
1¼	108		13 9/16	17 5/16	63.5	216.0	4.93	444.7
5/16	31		15¼	19⅜	24.2	82.6	5.54	17.0
⅜	36		15⅜	19½	28.1	96.0	5.59	190.7
7/16	41		15½	19⅝	32.0	109.3	5.64	226.0
½	46		15⅝	19 11/16	36.0	122.6	5.68	254.0
9/16	51		15¾	19¾	39.9	136.0	5.73	282.0
⅝	56		15⅞	19⅞	43.8	149.3	5.77	300.4
11/16	61	**G**	16	20	47.7	162.6	5.82	337.9
¾	66	14⅝	16⅛	20⅛	51.7	176.0	5.88	363.0
13/16	71		16¼	20¼	55.6	189.3	5.91	382.2
⅞	76		16⅜	20⅜	59.6	202.6	5.95	419.3
1	86		16⅝	20⅝	67.4	229.3	6.04	477.0
1⅛	96		16⅞	20⅞	75.3	256.0	6.13	522.3
1¼	106		17⅛	21	83.1	282.6	6.27	587.0
1⅜	116		17⅜	21¼	90.9	309.3	6.32	636.3

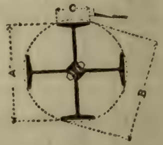
Least radius of gyration equals $D \times .3636$.

TABLE XXI.—SAFE LOADS IN TONS OF 2,000 LBS., LARIMER STEEL COLUMNS.

Allowed strain per square inch, $\begin{cases} 12{,}000 \text{ lbs. for lengths of 90 radii or under.} \\ 17{,}100 - 57\dfrac{l}{r} \text{ for lengths over 90 radii.} \end{cases}$

16″ COLUMNS.

Made of 15″ beams. Weight of filler bar 10.5 lbs. per foot. ¾″ rivets.



Weight per foot of single beam.	Weight per foot of column without fittings.	Sectional area of column.	Minimum radius of gyration.	Length of column in feet.			A	B	C
Lbs.	Lbs.	Sq.ins.	Inches.	32 and under.	36	40	A	B	C
60	131.25	38.4	4.36	232	220	208	16″	17 1/16″	6″
55	121.25	35.7	4.10	210	198	186	16 1/16	17 1/16	5 3/4
50	111.25	32.8	4.18	195	184	173	16	17	5 21/32
45	101.25	29.8	4.28	179	169	160	15 29/32	16 15/16	5 9/16
42	95.25	28.1	4.32	169	160	151	15 7/8	16 7/8	5 1/2

13″ COLUMNS.

Made of 12″ beams. Weight of filler bar 10.5 lbs. per foot. ¾″ rivets.

Lbs.	Lbs.	Sq.ins.	Inches.	24 and under.	28	32	A	B	C
45	101.25	29.7	3.40	178	170	158	13″	14 1/16″	5 3/8″
40	91.25	26.8	3.50	161	156	145	12 7/8	13 15/16	5 1/4
35	81.25	23.7	3.42	142	136	127	12 7/8	13 13/16	5 3/32
31.5	74.25	21.6	3.51	130	126	118	12 13/16	13 3/4	5

11″ COLUMNS.

Made of 10″ beams. Weight of filler bar 10.5 lbs. per foot. ¾″ rivets.

Lbs.	Lbs.	Sq.ins.	Inches.	22 and under.	24	28	A	B	C
30	71.25	20.8	2.84	123	118	108	10 7/8″	11 15/16″	4 13/16″
25	61.25	17.8	2.94	107	103	94	10 25/32	11 3/4	4 21/32

10″ COLUMNS.

Made of 9″ beams. Weight of filler bar 10.5 lbs. per foot. ¾″ rivets.

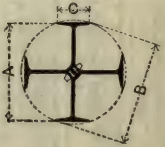
Lbs.	Lbs.	Sq.ins.	Inches.	20 and under.	24	28	A	B	C
25	61.25	18.1	2.57	106	97	87	9 7/8″	10 13/16″	4 15/32″
21	53.25	15.7	2.66	94	86	78	9 3/4	10 11/16	4 11/32

TABLE XXI.—SAFE LOADS IN TONS OF 2,000 LBS., LARIMER STEEL COLUMNS (*continued*).

Allowed strain per square inch, $\begin{cases} 12{,}000 \text{ lbs. for lengths of 90 radii or under.} \\ 17{,}100 - 57\dfrac{l}{r} \text{ for lengths over 90 radii.} \end{cases}$

9″ COLUMNS.

Made of 8″ beams. Weight of filler bar 6 lbs. per foot. 5/8″ rivets in web. 3/4″ rivets in flange.



Weight per foot of single beam.	Weight per foot of column without fittings.	Sectional area of column.	Minimum radius of gyration.	Length of column in feet.					
Lbs.	Lbs.	Sq.ins.	Inches.	18 and under.	20	24	A	B	C
22.75	52.0	15.4	2.37	91	86	77	9″	9 15/16″	4 3/16″
20.25	47.0	13.9	2.41	83	79	71	8 15/16	9 13/16	4 3/32
17.75	42.0	12.4	2.47	75	72	65	8 7/8	9 3/4	4

8″ COLUMNS.

Made of 7″ beams. Weight of filler bar 6 lbs. per foot. 5/8″ rivets.

Lbs.	Lbs.	Sq.ins.	Inches.	16 and under.	20	24	A	B	C
20.0	46.5	13.6	2.10	81	72	63	8″	8 7/8″	3 7/8″
17.5	41.5	12.1	2.12	72	64	56	7 15/16	8 3/4	3 3/4
15.0	36.5	10.6	2.16	64	57	50	7 27/32	8 5/8	3 21/32

7″ COLUMNS.

Made of 6″ beams. Weight of filler bar 6 lbs. per foot. 5/8″ rivets.

Lbs.	Lbs.	Sq.ins.	Inches.	14 and under.	16	20	A	B	C
14.75	36	10.5	1.85	62	58	50	6 15/16″	7 3/4″	3 7/16″
12.25	31	9.0	1.88	54	51	44	6 13/16	7 9/16	3 11/32

6″ COLUMNS.

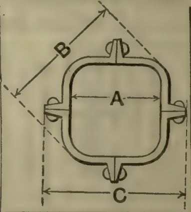
Made of 5″ beams. Weight of filler bar 6 lbs. per foot. 5/8″ rivets in web. 1/2″ rivets in flange.

Lbs.	Lbs.	Sq.ins.	Inches.	12 and under.	16	20	A	B	C
12.25	31	9.0	1.56	53	45	37	5 15/16″	6 3/4″	3 1/8″
9.75	26	7.5	1.62	45	39	32	5 13/16	6 9/16	3

TABLE XXII.—DIMENSIONS AND SAFE LOADS FOR NURICK STEEL COLUMNS.

(JONES & LAUGHLINS, PROPRIETORS.)

$p = 17{,}100 - 57\frac{l}{r}$ for lengths over 90 radii, and 12,000 lbs. for 90 radii and under.

Size of channels used.	Weight per foot of single channel.	Weight per foot of column without fittings.	Sectional area of column.	Minimum radius of gyration.	Safe load for length of 34 feet.	Safe load for length of 24 feet and under.			
Inches	Lbs.	Lbs.	Sq.ins.	Inches	Tons.	Tons.	A	B	C
15	45	183	53.8	6.58	323	323	$15\frac{3}{16}$	$21\frac{7}{16}$	$22\frac{15}{32}$
	40	163	47.9	6.50	287	287	$15\frac{1}{4}$	$21\frac{3}{8}$	$22\frac{5}{16}$
	33	135	39.6	6.45	237	237	$15\frac{7}{16}$	$21\frac{7}{32}$	$22\frac{7}{32}$
12	35	143	41.18	5.41	247	247	$12\frac{3}{16}$	$17\frac{3}{16}$	$18\frac{13}{16}$
	30	123	35.38	5.43	212	212	$12\frac{5}{16}$	$17\frac{1}{16}$	$18\frac{11}{16}$
	25	103	29.39	5.43	176	176	$12\frac{1}{2}$	$16\frac{15}{16}$	$18\frac{5}{8}$
	$20\frac{1}{2}$	85	24.12	5.45	145	145	$12\frac{9}{16}$	$16\frac{7}{8}$	$18\frac{7}{16}$
10	20	83	23.40	4.59	140	140	$10\frac{1}{2}$	$14\frac{1}{16}$	16
	15	63	17.84	4.53	107	107	$10\frac{5}{8}$	$13\frac{15}{16}$	$15\frac{13}{16}$
9	20	83	23.51	4.09	134	141	$9\frac{13}{32}$	$12\frac{3}{4}$	$14\frac{23}{32}$
	15	63	17.60	4.10	100	105	$9\frac{5}{16}$	$12\frac{5}{8}$	$14\frac{17}{32}$
	$13\frac{1}{4}$	53	15.55	4.11	89	93	$9\frac{5}{8}$	$12\frac{1}{2}$	$14\frac{1}{2}$
8	$16\frac{1}{4}$	68	19.29	3.57	102	116	$8\frac{13}{32}$	$11\frac{5}{16}$	$13\frac{9}{32}$
	$13\frac{3}{4}$	58	16.35	3.63	87	98	$8\frac{9}{16}$	$11\frac{3}{16}$	$13\frac{1}{4}$
	$11\frac{1}{4}$	48	13.40	3.65	72	80	$8\frac{5}{8}$	$11\frac{1}{8}$	$13\frac{1}{8}$
7	$14\frac{3}{4}$	62	17.20	3.21	85	103	$7\frac{13}{32}$	$9\frac{29}{32}$	$12\frac{1}{32}$
	$12\frac{1}{4}$	52	14.26	3.23	70	85	$7\frac{9}{16}$	$9\frac{3}{4}$	$11\frac{15}{16}$
	$9\frac{3}{4}$	42	11.39	3.25	56	68	$7\frac{5}{8}$	$9\frac{11}{16}$	$11\frac{13}{16}$
					24 ft.	18 ft. and under			
6	$10\frac{1}{2}$	44	12.48	2.76	69	75	$6\frac{9}{16}$	$8\frac{3}{8}$	$10\frac{21}{32}$
	8	34	9.52	2.85	54	57	$6\frac{23}{32}$	$8\frac{3}{16}$	$10\frac{9}{16}$
5	9	38	10.73	2.36	54	63	$5\frac{1}{2}$	7	$9\frac{1}{4}$
	$6\frac{1}{2}$	28	7.79	2.42	40	46	$5\frac{23}{32}$	$6\frac{13}{16}$	$9\frac{7}{32}$

$\frac{5}{8}''$ rivets in last four sections, $\frac{3}{4}''$ rivets in all others.

TABLE XXIII.—SAFE LOADS FOR GRAY COLUMNS.

Computed by the formula $W = 17,100 - 57\frac{l}{r}$.

12″ SQUARE COLUMNS.

No. of pieces.	Dimensions of angles.		Properties.			Safe loads in thousands of pounds.			
						Column lengths.			
	Length of legs.	Thick.	Area, square ins.	I.	r.	12 ft.	16 ft.	20 ft.	30 ft.
a 8	2½×2½	5/16	11.76	172	3.8	175	165	160	140
a 8	2½×2½	3/8	13.84	202	3.8	205	195	185	160
a 8	3 ×2½	5/16	12.96	198	3.9	195	185	175	150
a 8	3 ×2½	3/8	15.36	234	3.9	230	220	205	180
a 8	3 ×3	5/16	14.24	206	3.8	210	200	190	165
a 8	3 ×3	3/8	16.88	241	3.8	250	240	225	195
a 8	3 ×3	7/16	19.52	276	3.8	290	275	260	225
a 8	3 ×3½	5/16	15.44	209	3.7	230	215	205	180
a 8	3 ×3½	3/8	18.40	245	3.7	270	260	245	210
a 8	3 ×3½	7/16	21.20	282	3.7	315	300	280	245
a 8	3 ×3½	1/2	24.00	318	3.7	355	340	320	275
a 8	3 ×4	5/16	16.72	213	3.5	245	230	220	185
a 8	3 ×4	3/8	19.84	249	3.5	290	275	260	220
a 8	3 ×4	7/16	22.96	285	3.5	335	320	300	255
a 8	3 ×4	1/2	26.00	321	3.5	380	360	340	290
a 8	3 ×4	9/16	28.96	357	3.5	425	405	380	325
b 8	3 ×5	3/8	22.88	255	3.3	335	315	295	250
b 8	3 ×5	7/16	26.48	291	3 3	385	365	340	290
b 8	3 ×5	1/2	30.00	327	3.3	435	410	385	325
b 8	3 ×5	9/16	33.44	363	3.3	485	460	430	365

14″ SQUARE COLUMNS.

No. of pieces.	Length of legs.	Thick.	Area, square ins.	I.	r.	12 ft.	16 ft.	20 ft.	30 ft.
a 8	2½×2½	5/16	11.76	238	4.5	180	170	165	145
a 8	2½×2½	3/8	13.84	280	4.5	210	200	195	170
a 8	3 ×2½	5/16	12.96	274	4.6	195	190	180	160
a 8	3 ×2½	3/8	15.36	325	4.6	235	225	215	190
a 8	3 ×3	5/16	14.24	286	4.5	215	205	200	180
a 8	3 ×3	3/8	16.88	336	4.5	255	245	235	210
a 8	3 ×3	7/16	19.52	386	4.5	295	285	275	245
a 8	3 ×3½	5/16	15.44	293	4.3	235	225	215	190
a 8	3 ×3½	3/8	18.40	340	4.3	280	265	255	225
a 8	3 ×3½	7/16	21.20	386	4.3	320	305	295	260
a 8	3 ×3½	1/2	24.00	433	4.3	365	350	330	295
b 8	3 ×3½	9/16	26.72	479	4.3	405	385	370	330
b 8	3 ×3½	5/8	29.36	526	4.3	445	425	405	360
b 8	3 ×3½	11/16	32.00	572	4.3	485	465	445	395
b 8	3 ×3½	3/4	34.48	619	4.3	520	500	480	425
b 8	3 ×3½	13/16	36.96	666	4.3	560	535	515	455
a 8	3 ×4	5/16	16.72	300	4.2	250	240	230	205
a 8	3 ×4	3/8	19.84	348	4.2	300	285	275	240
a 8	3 ×4	7/16	22.96	396	4.2	345	330	315	280
a 8	3 ×4	1/2	26.00	444	4.2	390	375	360	315
b 8	3 ×4	9/16	28.96	491	4.2	435	420	400	350
b 8	3 ×4	5/8	31.84	539	4.2	480	460	440	385
b 8	3 ×4	11/16	34.72	587	4.2	525	500	480	420
b 8	3 ×4	3/4	37.52	635	4.2	565	540	520	455
b 8	3 ×4	13/16	40.24	683	4.2	605	580	555	490

a, tie-plates 8 inches wide, 2′ 6″ C. to C.
b, tie-plates 9 inches wide, 2′ 6″ C. to C.

SAFE LOADS FOR GRAY COLUMNS.

14″ SQUARE COLUMNS (*continued*).

	No. of pieces.	Dimensions of angles.		Properties.			Safe loads. Column lengths in thousands of pounds.			
		Length of legs.	Thick.	Area, square ins.	*I.*	*r.*	12 ft.	16 ft.	20 ft.	30 ft.
a	8	3 ×5	3/8	22.88	365	4.0	345	325	310	370
a	8	3 ×5	7/16	26.48	414	4.0	395	380	360	315
a	8	3 ×5	1/2	30.00	463	4.0	450	430	410	355
b	8	3 ×5	9/16	33.44	512	4.0	500	480	455	400
b	8	3 ×5	5/8	36.88	560	4.0	555	530	505	440
b	8	3 ×5	11/16	40.24	609	4.0	605	575	550	480
b	8	3 ×5	3/4	43.52	658	4.0	655	625	595	520
b	8	3 ×5	13/16	46.72	706	4.0	700	670	635	560
a	8	3½×3	5/16	15.44	320	4.5	235	225	215	190
a	8	3½×3	3/8	18.40	373	4.5	280	270	255	230
a	8	3½×3	7/16	21.20	425	4.5	320	310	295	265
a	8	3½×3	1/2	24.00	477	4.5	365	350	335	300
b	8	3½×3	9/16	26.72	529	4.5	405	390	375	335
b	8	3½×3	5/8	29.36	581	4.5	445	430	410	365
b	8	3½×3	11/16	32.00	633	4.5	485	470	450	400
b	8	3½×3	3/4	34.48	685	4.5	525	505	485	430
a	8	3½×3½	3/8	19.84	389	4.4	300	290	275	245
a	8	3½×3½	7/16	22.96	441	4.4	350	335	320	285
a	8	3½×3½	1/2	26.00	493	4.4	395	380	360	320
b	8	3½×3½	9/16	28.96	545	4.4	440	420	405	360
b	8	3½×3½	5/8	31.92	597	4.4	485	465	445	395
b	8	3½×3½	11/16	34.72	649	4.4	525	505	485	430
b	8	3½×3½	3/4	37.52	701	4.4	570	545	525	465
b	8	3½×3½	13/16	40.24	753	4.4	610	585	560	500
a	8	3½×5	3/8	24.40	407	4.0	365	350	330	290
a	8	3½×5	7/16	28.24	461	4.0	425	405	385	340
a	8	3½×5	1/2	32.00	515	4.0	480	460	435	385
b	8	3½×5	9/16	35.76	570	4.0	535	510	490	425
b	8	3½×5	5/8	39.36	624	4.0	590	565	535	470
b	8	3½×5	11/16	42.96	678	4.0	645	615	585	515
b	8	3½×5	3/4	46.48	732	4.0	700	665	635	555
b	8	3½×5	13/16	50.00	786	4.0	750	715	680	600
b	8	3½×5	7/8	53.36	840	4.0	800	765	730	640
a	8	3½×6	3/8	27.36	410	3.8	405	385	370	320
a	8	3½×6	7/16	31.76	468	3.8	475	450	430	370
b	8	3½×6	1/2	36.00	526	3.8	535	510	485	420
b	8	3½×6	9/16	40.24	584	3.8	600	570	540	470
b	8	3½×6	5/8	44.40	641	3.8	660	630	600	520
b	8	3½×6	11/16	48.48	698	3.8	725	690	655	565

a, tie-plates 8 inches wide, 2′ 6″ C. to C.
b, tie-plates 9 inches wide, 2′ 6″ C. to C.

SAFE LOADS FOR GRAY COLUMNS.

16″ SQUARE COLUMNS.

Tie-plates 9 inches wide, 2′ 6″ C. to C.

No. of pieces.	Dimensions of angles.		Properties.			Safe loads in thousands of pounds. Column lengths.			
	Length of legs.	Thick.	Area, square ins.	*I.*	*r.*	12 ft.	16 ft.	20 ft.	30 ft.
8	2½×2½	5/16	11.76	320	5.2	180	175	170	155
8	2½×2½	3/8	13.84	374	5.2	215	205	200	180
8	2½×2½	7/16	16.00	427	5.2	245	240	230	210
8	2½×2½	1/2	18.00	481	5.2	280	270	260	235
8	3 ×3	5/16	14.24	380	5.1	220	210	205	185
8	3 ×3	3/8	16.88	443	5.1	260	250	240	220
8	3 ×3	7/16	19.52	507	5 1	300	290	280	255
8	3 ×3	1/2	22.00	570	5.1	340	325	315	285
8	3 ×3	9/16	24.48	633	5.1	380	365	350	320
8	3 ×3	5/8	26.88	696	5.1	415	400	385	350
8	3½×3½	3/8	19.84	517	5.0	305	295	285	255
8	3½×3½	7/16	22.96	588	5.0	355	340	330	300
8	3½×3½	1/2	26.00	660	5.0	400	385	370	335
8	3½×3½	9/16	28.96	731	5.0	445	430	415	375
8	3½×3½	5/8	31.92	802	5.0	490	475	455	415
8	3½×3½	11/16	34.72	873	5.0	535	515	500	450
8	3½×3½	3/4	37.52	944	5.0	580	560	540	485
8	4 ×4	3/8	22.88	588	5.0	350	340	325	300
8	4 ×4	7/16	26.48	669	5.0	410	395	380	345
8	4 ×4	1/2	30.00	750	5.0	460	445	430	390
8	4 ×4	9/16	33.44	831	5.0	515	500	480	435
8	4 ×4	5/8	36.88	912	5.0	570	550	530	480
8	4 ×4	11/16	40.24	994	5.0	620	600	575	520
8	4 ×4	3/4	43.52	1075	5.0	670	650	625	565
8	4 ×4	13/16	46.72	1156	5.0	720	695	670	605
8	4 ×6	7/16	33.44	730	4.6	510	490	470	420
8	4 ×6	1/2	38.00	810	4.6	580	560	535	480
8	4 ×6	9/16	42.48	891	4.6	650	625	600	535
8	4 ×6	5/8	46.88	972	4.6	715	690	660	590
8	4 ×6	11/16	51.28	1053	4.6	785	755	720	645
8	4 ×6	3/4	55.52	1134	4.6	850	815	780	700
8	4 ×6	13/16	59.76	1215	4.6	915	880	840	755
8	4 ×6	7/8	63.92	1296	4.6	975	940	900	805

SAFE LOADS FOR GRAY COLUMNS.

16″ WALL COLUMNS.

Tie-plates 9 inches wide, 2′ 6″ C. to C.

No. of pieces.	Dimensions of angles.		Properties.			Safe loads in thousands of pounds. Column lengths.			
	Length of legs.	Thickness.	Area, square Ins.	*I.*	*r.*	12 ft.	16 ft.	20 ft.	30 ft.
6	2½×2½	5/16	8.82	111	3.6	130	120	115	100
6	2½×2½	3/8	10.38	120	3.6	150	145	135	115
6	2½×2½	7/16	12.00	139	3.6	175	165	160	135
6	2½×2½	1/2	13.50	167	3.6	200	185	180	150
6	3 ×3	5/16	10.68	131	3.5	155	150	140	120
6	3 ×3	3/8	12.66	157	3.5	185	175	165	140
6	3 ×3	7/16	14.64	179	3.5	215	205	190	165
6	3 ×3	1/2	16.50	200	3.5	240	230	215	185
6	3 ×3	9/16	18.36	222	3.5	270	255	240	205
6	3 ×3	5/8	20.16	244	3.5	295	280	265	225
6	3½×3½	3/8	14.88	188	3.5	220	205	195	165
6	3½×3½	7/16	17.22	215	3.5	255	240	225	190
6	3½×3½	1/2	19.50	241	3.5	285	270	255	220
6	3½×3½	9/16	21.72	268	3.5	320	300	285	245
6	3½×3½	5/8	23.94	294	3.5	350	335	315	270
6	3½×3½	11/16	26.04	321	3.5	385	365	340	290
6	3½×3½	3/4	28.14	347	3.5	415	395	370	315
6	4 ×4	3/8	17.16	221	3.5	250	240	225	190
6	4 ×4	7/16	19.86	252	3.5	290	275	260	225
6	4 ×4	1/2	22.50	283	3.5	330	315	295	250
6	4 ×4	9/16	25.08	314	3.5	370	350	330	280
6	4 ×4	5/8	27.66	345	3.5	405	385	365	310
6	4 ×4	11/16	30.18	376	3.5	445	420	400	340
6	4 ×4	3/4	32.64	407	3.5	480	455	430	365
6	4 ×4	13/16	35.04	439	3.5	515	490	460	395
6	4 ×6	7/16	25.08	279	3.3	365	345	325	270
6	4 ×6	1/2	28.50	311	3.3	415	390	370	310
6	4 ×6	9/16	31.86	343	3.3	465	440	410	345
6	4 ×6	5/8	35.16	375	3.3	510	485	455	380
6	4 ×6	11/16	38.46	407	3.3	560	530	500	415
6	4 ×6	3/4	41.64	440	3.3	605	570	540	450
6	4 ×6	13/16	44.82	472	3.3	655	615	580	485
6	4 ×6	7/8	47.94	504	3.3	700	660	620	520

SAFE LOADS FOR GRAY COLUMNS.

$10\frac{1}{2}''$ CORNER COLUMNS.

	No. of pieces.	Dimensions of angles.		Properties.			Safe loads in thousands of pounds.			
		Length of legs.	Thickness.	Area, square inches.	*I.*	*r.*	Column lengths.			
							12 ft.	16 ft.	20 ft.	30 ft.
a	4 1	3½×3 3 ×3	5/16 3/8	9.83	118	3.5	145	135	130	110
a	4 1	3½×3 3 ×3	3/8 3/8	11.31	139	3.5	165	155	150	125
a	4 1	3½×3 3 ×3	7/16 1/2	13.35	145	3.5	195	185	175	150
a	4 1	3½×3 3 ×3	1/2 1/2	14.75	159	3.5	215	205	195	165
b	4 1	3½×3 3 ×3	9/16 5/8	16.72	172	3.5	245	230	220	185
b	4 1	3½×3 3 ×3	5/8 5/8	18.04	186	3.5	265	250	235	200
b	4 1	3½×3 3 ×3	11/16 5/8	19.36	199	3.5	285	270	255	215
b	4 1	3½×3 3 ×3	3/4 5/8	20.60	213	3.5	305	285	270	230
a	4 1	3½×3½ 3 ×3	3/8 3/8	12.03	139	3.4	175	165	155	130
a	4 1	3½×3½ 3 ×3	7/16 1/2	14.23	158	3.4	205	195	185	155
a	4 1	3½×3½ 3 ×3	1/2 1/2	15.75	177	3.4	230	215	205	175
b	4 1	3½×3½ 3 ×3	9/16 5/8	17.84	196	3.4	260	245	230	195
b	4 1	3½×3½ 3 ×3	5/8 5/8	19.32	215	3.4	280	265	250	210
b	4 1	3½×3½ 3 ×3	11/16 5/8	20.72	234	3.4	305	285	270	230
b	4 1	3½×3½ 3 ×3	3/4 5/8	22.12	253	3.4	325	305	290	245
b	4 1	3½×3½ 3 ×3	13/16 5/8	23.48	272	3.4	345	325	305	260
a	4 1	3½×4 3 ×3	3/8 3/8	12.79	141	3.3	185	175	165	140
a	4 1	3½×4 3 ×3	7/16 1/2	15.11	160	3.3	220	205	195	165
a	4 1	3½×4 3 ×3	1/2 1/2	16.75	180	3.3	245	230	215	180
b	4 1	3½×4 3 ×3	9/16 5/8	18.96	199	3.3	275	260	245	205
b	4 1	3½×4 3 ×3	5/8 5/8	20.56	219	3.3	300	280	265	220
b	4 1	3½×4 3 ×3	11/16 5/8	22.08	238	3.3	320	305	285	240
b	4 1	3½×4 3 ×3	3/4 5/8	23.60	258	3.3	345	325	305	255
b	4 1	3½×4 3 ×3	13/16 5/8	25.08	277	3.3	365	345	325	270
a	4 1	3½×5 3 ×3	3/8 3/8	14.31	145	3.2	205	195	180	150
a	4 1	3½×5 3 ×3	7/16 3/8	16.23	165	3.2	235	220	205	170
a	4 1	3½×5 3 ×3	1/2 1/2	18.75	185	3.2	270	255	240	200

a, tie-plates 8 inches wide, 2′ 6″ C. to C.
b, tie-plates 9 inches wide, 2′ 6″ C. to C.

KINDS OF COLUMNS USED IN THE PRINCIPAL OFFICE BUILDINGS OF CHICAGO AND NEW YORK.

CHICAGO.

Architect.	Building.	No. of stories.	Kind of column.
W. L. B. Jenney	Manhattan	16	Cast
"	"The Fair"	9	Z-bar
"	Y. M. C. A.	13	Z-bar
"	Isabella	10	Z-bar
Jenney & Mundie	New York Life	12	Steel; plates and angles
"	Fort Dearborn	12	Channels and plates
Holabird & Roche	Tacoma	13	Cast
"	Pontiac	14	Z-bar
"	Venetian	13	Z-bar
"	Monadnock Block, new part	17	Z-bar
"	Old Colony	17	Z-bar and Phœnix
"	Champlain	15	Z-bar
"	Marquette	16	Z-bar
Adler & Sullivan	Auditorium	10 & 17	Cast
"	Schiller Theatre	13 & 17	Z-bar and Phœnix
"	Stock Exchange	13	Z-bar
Burnham & Root	Rookery	12	Cast
"	Woman's Temple	13	Z-bar
"	Masonic Temple	20	Plates and angles
"	Ashland	16	Z-bar
D. H. Burnham & Co.	Reliance	15	Gray
"	Fisher	18	Gray
"	Great Northern Theatre	16	Gray
Henry Ives Cobb	Title & Trust	16	Phœnix
"	Owings	14	Cast

NEW YORK.

Architect.	Building.	No. of stories.	Kind of column.
Bruce Price	American Surety	21	Angles and plates—Z-bar
Kimball & Thompson	Manhattan Life Ins.	18	Cast, 5 stories: plates and angles above
Geo. B. Post	Meyer, Jonasson	14	Plates and angles
"	Havemeyer (Cortland St.)	15	Plates and angles
"	St. Paul Building	26	Plates and angles
"	New York World	22	Phœnix
"	Union Trust	10	Phœnix
Harding & Gooch	Postal Telegraph	14	Cast
"	Dunn Building	15	Phœnix
"	Park Row Building	29	Plates and angles
"	Commercial Cable	20	Phœnix
R. H. Robertson	American Tract Society	20	Riveted
H. J. Hardenberg	Hotel Waldorf	12	Cast

FORMULAS FOR STEEL COLUMNS AND STRUTS,

GIVEN IN VARIOUS BUILDING LAWS, AND RECOMMENDED BY LEADING ENGINEERS.

p = working stress in pounds per square inch of cross-section. l = length in inches. r = least radius of gyration.

Chicago Building Law:

For columns more than 60 radii in length, $p=17{,}000-\frac{60l}{r}$.

For columns less than 60 radii in length, $p=13{,}500$ lbs.

Greater New York Law (1899):

$$p=15{,}200-58\frac{l}{r}.$$

Boston Law:

$p=12{,}000$ lbs. reduced by approved modern formulæ.

Buffalo Building Law (1896):

For columns more than 90 radii in length, $p=17{,}100-57\frac{l}{r}$.

For columns less than 90 radii in length, $p=12{,}000$ lbs.

Denver Building Ordinance:

Same as Buffalo.

Carnegie Steel Co.; Jones & Laughlins:

$$p=17{,}100-57\frac{l}{r} \text{ for lengths over 90 radii.}$$

$$p=12{,}000 \text{ for lengths of 90 radii and under.}$$

Passaic Rolling Mill Co. Geo. H. Blakeley, C.E.

$$\text{Z-bar and box columns.}\left\{\begin{array}{l} p=12{,}000 \text{ for lengths up to 50 radii.} \\ p=15{,}000-57\frac{l}{r} \text{ for lengths over 50 radii.}\end{array}\right.$$

$$\text{Steel Struts.}\left\{\begin{array}{l} 12{,}000 \text{ for lengths up to 30 radii.} \\ 13{,}500-50\frac{l}{r} \text{ for lengths over 30 radii.}\end{array}\right.$$

Charles Evan Fowler, C.E. General specifications for steel roofs and buildings:

For struts with flat and fixed ends, $p=12{,}500-41\tfrac{2}{3}\frac{l}{r}$.

Theodore Cooper, M. Am. Soc. C. E. General specifications for iron and steel railroad bridges and viaducts:

For chords. $\begin{cases} p= \ 8{,}000-30\dfrac{l}{r} \text{ for live load stresses.} \\ p=16{,}000-60\dfrac{l}{r} \text{ for dead load stresses.} \end{cases}$

For posts. $\begin{cases} p= \ 7{,}000-40\dfrac{l}{r} \text{ for live load stresses.} \\ p=14{,}000-80\dfrac{l}{r} \text{ for dead load stresses.} \\ p=10{,}000-60\dfrac{l}{r} \text{ for wind load stresses.} \end{cases}$

Formula proposed by Mr. Thomas H. Johnson, M. Am. Soc. C. E.:

For mild steel. $\begin{cases} \text{Hinged ends, } p=13{,}125-55\dfrac{l}{r}. \\ \text{Flat ends, } p=13{,}125-45\dfrac{l}{r}. \end{cases}$

Mr. Joseph K. Freitag, in commenting on the allowed working stress for steel columns in buildings says:

"The writer believes that with the use of a mild steel of an ultimate strength of from 65,000 to 68,000 lbs. per square inch, 15,000 or 16,000 lbs. per. square inch may safely be used for all concentric dead, live, and wind loads combined (with an additional allowance for eccentric loading), provided that the wind pressure is taken at not less than 30 lbs. per square foot and that the live loads on the floors are assumed as required by municipal building laws."

CHAPTER XV.

GENERAL PRINCIPLES OF THE STRENGTH OF BEAMS, AND STRENGTH OF STEEL BEAMS.

STEEL-BEAM BOX GIRDERS—FRAMING AND CONNECTING OF STEEL BEAMS.

By the term "beam" is meant any piece of material which supports a load whose tendency is to break the piece across, or at right angles to, the fibres, and which also causes the piece to bend before breaking. A simple beam is one which rests upon supports at both ends. When a beam is supported at its centre it is a cantilever beam, or if a part of a beam projects from a wall or beyond a support, the projecting part is called a cantilever. In a simple beam the lower part is in tension and the upper part in compression; in a cantilever beam the reverse is the case.

When a transverse load of any kind is applied to any beam it will cause the beam to bend by a certain amount, and as it is impossible to bend a piece of any material without stretching the fibres on the outer side, and compressing the fibres on the inner side, the bending of the beam will produce tension in the stretched side and compression of the fibres of the opposite side. Between the stretched and compressed fibres is a neutral surface which is unchanged in length. From experiments it has been found that the amount of elongation or shortening of any fibre is directly proportional to its distance from the neutral surface; hence, if the elastic limit be not surpassed, the stresses are also proportional to their distance from the neutral surface. The line where the neutral surface would cut through the side or cross-section of the beam is called the *neutral axis*. Within the safe strength of the material the neutral axis passes through the *centre of gravity* of the cross-section of the beam for all materials.

To determine the strength of any beam to resist the effects of any load, or series of loads, we must determine two things: first,

the destructive force tending to bend and break the beam, which is called the "*bending-moment*"; and, second, the combined resistance of all the fibres of the beam to being broken, which is called the "*moment of resistance.*"

The methods for finding the bending-moments for any load, or series of loads, have been given in Chap. IX. The moment of resistance is equal to the "moment of resistance area" or "section modulus," multiplied by the strength of the material. Formulas for finding the section modulus of common shapes are given in Chap. X., and the value, R, for the section modulus of merchant shapes of structural steel is given in the tables, pages 296 to 315.

The "*coefficient of strength*" usually given in tables of steel beams is the maximum distributed load that a beam of one foot span would support without producing a fibre stress exceeding the safe limit, generally 16,000 lbs. As the strength of a beam is inversely as its span, the safe load for any span may be obtained by dividing the coefficient by the span in feet.

Now, that a beam shall just be able to resist the load, and not break, we must have a condition where the bending-moment in the beam is equal to the section modulus multiplied by the strength of the material. That the beam may be abundantly *safe* to resist the given load, this product must be several times as great as the bending-moment; and the ratio in which this product exceeds the bending-moment, or in which the breaking-load exceeds the safe load, is known as the "factor of safety."

By "the strength of the material" is meant a certain constant quantity, which is determined by experiment, and which is known as the "Modulus of Rupture." Of course this value is different for each different material. The following table contains the values of this constant divided by the factor of safety, for most of the materials used in building construction. The section modulus multiplied by these values will give the *safe resisting-power* of the beam.

The term "Modulus of Rupture" is now seldom seen in the various handbooks published by the rolling-mill companies, the term "fibre stress" being used instead. The two terms, however, are synonymous.

The following values of S for wrought iron and steel are one-fourth that for the breaking-loads; for cast iron, one-sixth; for wood, one-third; and for stone, one-sixth. The constants for wood are based upon tests made at the Massachusetts Institute of

MODULUS OF RUPTURE (*S*) FOR SAFE STRENGTH.*

Material.	Value of S, in lbs.	Material.	Value of S, in lbs.
Cast iron	5,544	American White pine	1,080
Wrought iron	12,000	American yellow pine	1,800
Steel	16,000	American spruce	1,260
		Oregon pine	1,620
American ash	2,000		
American red beech	1,800	Bluestone flagging (Hudson River)	450
American yellow birch	1,620		
American white cedar	1,000	Cement, 1 3:6	370
American elm	1,400	Granite, average	300
Chestnut	1,080	Limestone	250
Hemlock	990	Marble	300
American white oak	1,350	Sandstone	150to200
		Slate	900

* For a comparison of values given in different Building Laws see last pages of Chapter XVI.

Technology upon full-size timbers of the usual quality found in buildings. The figures given in the above table are believed to be amply safe for beams in floors of dwellings, public halls, roofs, etc.; but for floors in mills, and warehouse floors, the author recommends that not more than two-thirds of the above values be used. The safe loads for the steel sections given in the following tables are all computed on the value of 16,000 lbs for S. For angles, tees, and deck-beams it was customary, previous to 1898, to use a somewhat lower value for S, about 12,000 lbs., on account of the section not being symmetrical. All but one of the steel companies now use 16,000 lbs. for all shapes, and the author has therefore revised the tables in this book to correspond; but these full loads should be used with caution, and reduced under the conditions noted. For riveted steel girders the value of S is generally taken at 13,000 lbs.

There are certain cases of beams which most frequently occur in building construction, for which formulas can be given by which the safe loads of the beams may be determined directly; but it often happens that we may have either a regularly shaped beam irregularly loaded, or a beam of irregular shape but with a common method of loading. For such cases it is impossible to give tables for strength, as each case must be computed by determining either the section modulus required to resist the bending-moment, or the greatest bending-moment that may be allowed for a given value of R (section modulus).

The general formula for any beam under any system of loading is as follows:

Greatest bending-moment (inch-lbs.) = section modulus $\times S$, (*a*)
or

$$\text{Section modulus } (R=) \frac{\text{bending moment (inch-lbs.)}}{S}. \qquad (b)$$

If the bending-moment is computed in *foot-pounds,* these formulas become:

$$\text{Greatest bending-moment} = \frac{\text{section modulus} \times S}{12}, \qquad (c)$$

or

$$\text{Section modulus } (R) = \frac{12 \times \text{bending-moment}}{S}. \qquad (d)$$

By substituting for the bending-moment its value in terms of the load and span, the following formulæ may readily be deduced which apply to *any shape* of beam.

FORMULÆ FOR STRENGTH OF BEAMS FOR DIFFERENT CONDITIONS OF SUPPORT AND LOADING.

R = section modulus; S = safe modulus of rupture, or fibre stress in pounds (p. 499).

W = total load on beam in pounds.

L = span in feet.

C = coefficient of strength given in tables.

Values for R for the various shapes and sizes of structural steel bars will be found in the tables on pages 296–315.

CASE 1.—*Beams fixed at one end and loaded at the other* (Fig. 1).

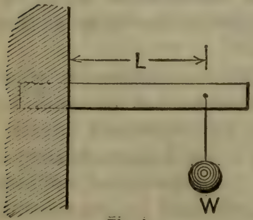


Fig. 1.

Safe load in pounds =

$$\frac{R}{12L} \times S, \quad \text{or} \quad \frac{C^*}{8L}. \qquad (1)$$

$$R = \frac{12W \times L}{S}. \qquad (1A)$$

EXAMPLE.—A steel T-bar is fixed at one end in a brick wall, and loaded at the other end with 600 lbs., the distance L being 4 ft. What size bar should be used to support the load with safety?

* When C is given in tons, safe load will be tons.

Ans. We will allow 12,000 lbs. for the value of S; then $R = \frac{12 \times 600 \times 4}{12,000} = 2.4$. Now we must ascertain what size T-bar has a section modulus equal to 2.4. Looking in the table giving the properties of tees (p. 313, col. VII.), we find 2.43 opposite a $4 \times 5 \times \frac{3}{8}$ tee, and 2.55 opposite a $4 \times 4\frac{1}{2} \times \frac{1}{2}$ tee; hence either size will have sufficient strength, the first, however, being the cheapest, as it weighs the least.

For an I-beam we could have used 16,000 lbs. for S; then R would equal $\frac{12 \times 600 \times 4}{16,000} = 1.8$, which would permit of using a 3-inch $6\frac{1}{2}$-lb. beam.

CASE 2.—*Beams fixed at one end, loaded with uniformly distributed load* (Fig. 2).

Safe load in pounds=

$$\frac{R}{6L} \times S, \quad \text{or} \quad \frac{C^*}{4L}. \tag{2}$$

$$R = \frac{6W \times L}{S}. \tag{2A}$$

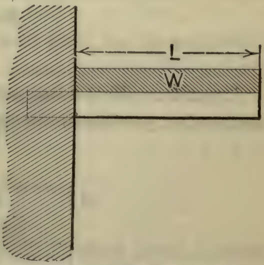


Fig. 2.

CASE 3.—*Beams supported at both ends, loaded at middle* (Fig. 3).

$$\text{Safe load in pounds} = \frac{R}{3L} \times S, \quad \text{or} \quad \frac{C^*}{2L}. \tag{3}$$

$$R = \frac{3W \times L}{S}. \tag{3A}$$

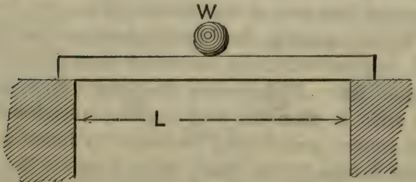


Fig. 3.

* When C is given in tons, safe load will be tons.

CASE 4.—*Beams supported at both ends, load uniformly distributed* (Fig. 4).

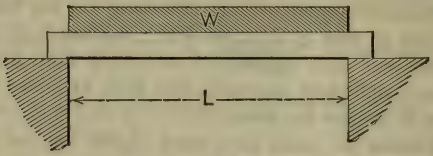


Fig. 4.

$$\text{Safe load in pounds} = \frac{2R}{3L} \times S, \quad \text{or} \quad \frac{C^*}{L}. \tag{4}$$

$$R = \frac{3W \times L}{2S}. \tag{4A}$$

CASE 4A.—*Beams supported at both ends, with a distributed load over only a portion of the span, as in Fig. 4a.*

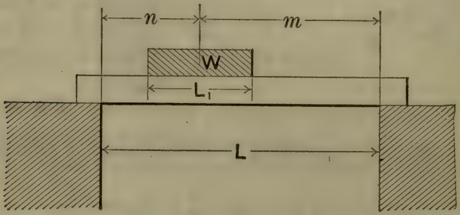


Fig. 4a.

In this case the load is generally given, and the problem will be to determine the size of the beam. This can be accurately done only by computing the bending-moment as explained in Chapter IX., and substituting the value thus found in formulas (*b*) or (*d*), page 500. If, however, the length L_1 is very short in comparison with L, then the load may be considered as concentrated at its centre, and R may be found by formula ($3A$) if the load is at the centre of the beam, or by formula ($5A$) if the load is at one side of the centre. The error will be on the safe side.

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* When C is given in tons, safe load will be tons.

CASE 5. *Beams supported at both ends, loaded with concentrated load not at centre* (Fig. 5).

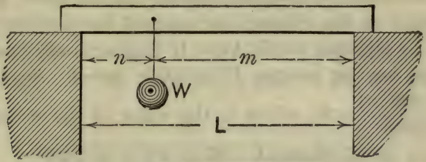

Fig. 5.

$$\text{Safe load in pounds} = \frac{R \times L}{12 \times m \times n} \times S. \qquad 5)$$

$$R = \frac{12W \times m \times n}{L \times S}, \; m \text{ and } n \text{ being measured in feet.} \qquad (5A)$$

EXAMPLE.—A steel I-beam of 20 feet span has to support a concentrated load of 24,000 lbs. at a distance of 6 feet from one support. What must be the size and weight of the beam?

Ans. In this case $W = 24{,}000$, $L = 20$, $n = 6$, $m = 14$, and we will allow 16,000 lbs. for S.

Then $$R = \frac{12 \times 24{,}000 \times 6 \times 14}{20 \times 16{,}000} = 75.6.$$

Looking down column VII. of the Properties of Steel I-beams (p. 296), we find that the nearest value (above) to 75.6 is 81.2 for a 15-inch 60-lb. beam, and 117.0 for a 20-inch 65-lb. beam, or we might use two 12-inch 35-lb. beams. The 15-inch 60-lb. beam would, however, be the cheapest beam to use, although the 20-inch beam would deflect much less under the load.

CASE 6.—*Beams supported at both ends, loaded with* W *pounds, at the same distance* m *from each end* (Fig. 6).

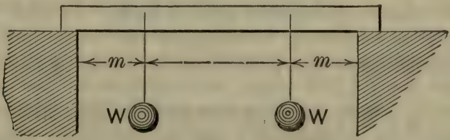

Fig. 6.

$$\text{Safe load } W\text{, in pounds, at each point} = \frac{R}{12 \times m} \times S, \qquad (6)$$

or $$R = \frac{12 \times W \times m}{S}. \qquad (6A)$$

EXAMPLE.—A 12-inch standard steel channel of 12 ft. span supports the ends of two 10-inch beams 4 ft. from each support. Each 10-inch beam is designed to carry 16,000 lbs. What should be the weight of the channel to support the beams?

Ans. The channel would have to support only one-half of the load on the beams, whence $W=8{,}000$; $m=4$; $S=16{,}000$; and $R=\frac{12\times 8{,}000\times 4}{16{,}000}=24$, which is the value of the section modulus of a 12-inch 25-lb. channel. (Col. VII., p. 298.)

It will be noticed that in formulas (6) and (6A) the span of the beam is not taken into account, and if the beam itself had no weight it would make no difference in the fibre strains how far apart the loads W are placed. In reality, however, steel beams do weigh considerable, and to be absolutely correct an example such as the above should be calculated for the weight of the beam, as well as for the weights W. The weight of the beam would, of course, be a distributed load, and to be absolutely correct the maximum bending-moment on the beam should be found graphically in a manner similar to that explained on the lower half of page 272, and the value of R computed by formula (7). Where the loads, however, are spaced so as to divide the beam into three equal parts, as in the last example, one-third of the weight of the beam may be added to W with sufficient accuracy; thus, the weight of the channel in the above example between supports would be 12×25, or 300 lbs., and W should be taken at 8,100 lbs., which would give a value for R of 24.1.

Generally there is a sufficient factor of safety in the loads allowed to offset the slight effect produced by the weight of the beam; but if the full load assumed is likely to be imposed on the beam, then allowance must be made for the weight of the beam itself.

CASE 7.—*Beam loaded with several loads.*—In such a case as this it will be necessary to compute the maximum bending-moment on the beam and proportion the beam by the formula

$$R=\frac{\text{max. bending-moment (ft.-lbs.)}\times 12}{S}. \qquad (7)$$

EXAMPLE.—A steel beam girder is to be used to support a brick wall, 16 inches thick and weighing 138,000 lbs., over an opening 22 ft. wide. The girder must also support the ends of four 10-inch floor-beams spaced as in Fig. 7, each beam being

assumed to carry 16,000 lbs.; what should be the size of the beams forming the girder?

Ans. The first step should be to determine on the allowance for the weight of the girder. The total load on the girder = $138,000 + 4 \times 8,000$ (one-half the load on each beam) = 170,000 lbs., or 85 tons. As we must use a pair of beams the load on

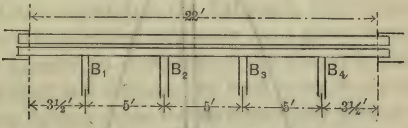

Fig. 7.

each beam will be 42.5 tons. For our present purpose we may consider the entire load as distributed, and from the table of strength of I-beams, we find that to support 42.5 tons with a span of 22 feet would require a 24-inch 85-lb. beam. Our girder would then weigh between supports $2 \times 85 \times 22 = 3,740$ lbs., or say 4,000 lbs. Adding this to the weight of the wall we have for the total distributed load 142,000 lbs. The next step will be to determine the maximum bending-moment.

By the formulas given in Chapter IX. we find the bending-moments for the various loads to be as follows:

For weight of wall, $$M = \frac{22 \times 142,000}{8} = 390,500 \text{ ft.-lbs.}$$

For beam B_1, $$M_1 = \frac{8,000 \times 3\frac{1}{2} \times 18\frac{1}{2}}{22} = 23,545 \text{ ft.-lbs.}$$

For beam B_2, $$M_2 = \frac{8,000 \times 8\frac{1}{2} \times 13\frac{1}{2}}{22} = 41,727 \text{ ft.-lbs.}$$

The beams being spaced symmetrically from the centre of the span, the bending-moments for B_3 and B_4 will be equal to those of B_2 and B_1 respectively. Platting the bending-moments to a scale, in the manner explained on pages 272, 273, we obtain the diagram shown in Fig. 8, the greatest bending-moment being the line M_0, which scales 485,000 lbs. Multiplying this moment by 12 and dividing by S (= 16,000 lbs.), we have 363.7 as the value for R for both beams, or 181.8 for one beam. From column VII., p. 296, Properties of I-beams, we find that a 24-inch 90-lb. beam has a section modulus of 186.6, therefore two 90-lb. beams will just answer. The maximum load for a single beam is 50.9

tons, which is greater than the load we wish to carry, so that there is no danger of the web buckling. The two beams should be securely bolted together with separators near each connection of beams B_1, B_2, B_3, B_4, and at each end of the girder.

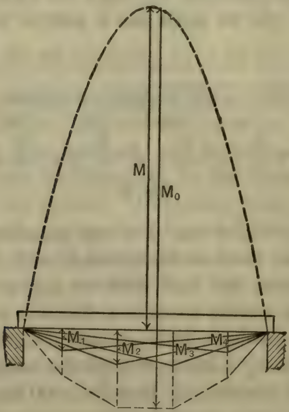

Fig. 8.

The method above indicated applies to any method of loading. the only difference in the calculation being in determining the bending-moment.

Inclined Beams.—The strength of beams inclined to the horizon may be computed, with sufficient accuracy for most purposes, by using the formulas given for horizontal beams, and taking the horizontal projection of the beam as its span.

Steel Beams.

Practically the only materials used in structural work for beams, at the present day, are wood and steel. Wooden beams being always rectangular in cross-section, the general formula can be much simplified by substituting for R its value in terms of the breadth and depth of the beam. Formulas for wooden beams will therefore be found in Chapter XVI. Cast iron is also occasionally used for beams or lintels, but as this material is much stronger to resist compression than tension, the beam must be of a special shape in order to use the material to advantage. The

strength of cast-iron beams is therefore considered under a special heading in Chapter XVI. Formulas for concrete-steel beams are given in Chapter XXIV.

Since 1893, steel beams have practically superseded wrought-iron beams, and the latter are now seldom, if ever, used. The same formulas apply to wrought-iron as to steel beams, however, by simply changing the value of S. Any shape of rolled steel may be used as a beam, but the I shape is the most economical, as it possesses the greatest resistance for a given weight of metal. Next to the I-beam, in economy, is the channel, then the deck-beam, and angles and tees are the least economical of all shapes. The following figures show the safe load per pound of steel, for the various sections, for a 10-foot span; the same ratio would hold for other spans.

10″ I-beam	10″ channel	10″ deck-beam	4×6 angle	4×5 tee
104	94.6	83.0	28.7	21.6

Deepest Beams Stiffest and most Economical.— The strength of a wrought-iron, wooden, or steel beam of rectangular shape varies as the *square* of the *depth*, and directly as the breadth; hence the deeper beam will have the greatest strength in proportion to its sectional area. With I-beams this rule in regard to the square of the depth does not hold strictly true, on account of the variation in the sections, but it is approximately true.

It therefore follows that, for any given span, it is more economical, where other conditions will permit, to use deep beams spaced farther apart in floors, or to use one deep beam in place of two shallower beams. Thus if we wished to support a distributed load of 39 tons with a 16-ft. span, we might use one 20-inch 70-lb. beam, two 15-inch 42-lb. beams, or three 12-inch 40-lb. beams, but the 20-inch beam would weigh only 1,190 lbs. (allowing for 6-inch bearings) as compared with 1,428 lbs. for the 15-inch beams and 2,040 lbs. for the 12-inch beams, besides the saving in bolts and separators.

Light beams are also more economical than heavy beams *of the same depth*, except when the span is so short that the safe load is governed by the resistance of the web to buckling, in which case the heavy beams are the more economical.

Maximum Safe Load for Steel Beams.—All beams are subject in a greater or less degree to three kinds of strains. The most destructive of these is generally the bending-moment, which has already been considered. The second kind is that

which tends to shear the beam, or make one part slide on the other vertically. This strain, however, seldom needs to be considered except in the case of riveted girders, and short beams with very thick webs. The third strain is that which tends to cause the web of the beam to buckle, and for steel beams, where the span is very short in proportion to the depth of the beam, the resistance of the web to buckling generally determines the maximum load that the beam will support, without stiffening the webs.

In the tables giving the safe loads for I-beams, channels, and deck-beams, the column headed Max. Load gives the greatest load that should be put on the beam, no matter how short the span, unless the web is stiffened by riveting plates or angles to the web. This load may be either distributed or concentrated *at the centre*, provided it *does not exceed the safe load as determined by the bending-moment.* In the tables giving safe loads for I-beams and channels the loads to the *left* of the dotted line exceed the maximum distributed loads, and hence the beam should not be used for those spans, unless stiffening is resorted to. For concentrated loads very much shorter spans may be used than for distributed loads. Thus for a 24-inch 80-lb. beam the shortest span for a full distributed load is 25 feet, while for a concentrated load at the centre 12 feet would be the shortest span. In using the tables for safe loads, therefore, the maximum load should always be considered. The maximum loads in the table are such that the greatest shear will not exceed 10,000 lbs. per. sq. inch, nor the total load exceed that obtained by the formula

$$\text{Max. load in tons} = \frac{8 \times d \times t}{1 + \frac{2d^2}{3{,}000t^2}}, \qquad (8)$$

in which d = depth of beam and t = thickness of web, both in inches.

This formula is that used by the engineers of the Pencoyd Iron Works, and gives results considerably less than formulas used by some of the other steel companies.

It is based on Gordon's formula for long columns, considering the length of the column equal to the diagonal depth of the beam, and using 8,000 lbs. as the unit strength of the material instead of ten or twelve thousand pounds, as used for columns and the webs of riveted girders.

In comparing this formula with formulas giving the safe resistance to buckling, it should be remembered that these formulas give the *maximum shear*, and that the maximum shear is only one-half of the safe load, when the load is either distributed or concentrated at the centre.

For beams unsymmetrically loaded the maximum shear should not exceed one-half of the maximum load.

Short lengths of beams used as blocking or bolsters should not be loaded to more than one-half the maximum load, given in the tables.

Lateral Strength.—As has been stated, the effect of the bending-moment on a beam is to produce compression in the top flange and tension in the lower flange; the top flange therefore becomes in effect a strut, and when *its length exceeds* 20 *times its width* the compression tends to deflect the beam sideways. Provision should therefore be made for bracing the beam sideways at intervals not exceeding 20 times the width of the flange. Floor-beams are generally sufficiently braced by the filling between the beams and by the tie-rods. In the case of a pair of beams bolted together the total width may be taken in determining the maximum length.

In cases where it is not practical to support the beams sideways, the loads given in the tables should be reduced as indicated in the following table:

BEAMS WITHOUT LATERAL SUPPORT.

Length of beam.	Proportion of tabular load forming greatest safe load.
20 times flange width	Whole tabular load.
20 to 30 " " "	9/10 " "
30 to 40 " " "	8/10 " "
40 to 50 " " "	7/10 " "
50 to 60 " " "	6/10 " "
60 to 70 " " "	5/10 " "

EXAMPLE.—What is the maximum distributed load that should be allowed for a 15-inch 42-lb. beam 25-foot clear span, the beam being unsupported sideways?

Ans. The flange width of this beam (see table of properties of standard beams) is $5\frac{1}{2}$ in. 25 ft. (300 in.) $\div 5\frac{1}{2} = 54$. We should therefore use only .65 of the load given in the table, or 8.12 tons. From this the weight of the beam (1,050 lbs.) should also be subtracted, reducing the maximum safe load to 7.6 tons.

Deflection of Steel Beams.—The principles and general formula for the deflections of beams are given in Chapter XVIII., and the deflection of any beam under any load may be found by the formula there given. A shorter and sufficiently accurate method of finding the deflection of steel beams under the safe loads given in the tables is afforded by the following table, taken from the "Pocket Companion" of the Carnegie Steel Company:

DEFLECTION COEFFICIENTS FOR SYMMETRICAL SHAPES GIVEN IN 64THS OF AN INCH.

Coefficient index.	Distance between supports in feet.								
	6	8	10	12	14	16	18	20	22
C	38.0	68.0	106.0	152.5	208.0	271.0	343.0	424.0	513.0
C'	30.0	53.0	83.0	119.0	162.0	212.0	268.0	331.0	400.5

Coefficient index.	Distance between supports in feet.								
	24	26	28	30	32	34	36	38	40
C	610.0	716.0	830.5	953.0	1085.0	1225.0	1373.0	1530.0	1695.0
C'	477.0	559.0	649.0	748.0	847.0	957.0	1073.0	1195.0	1324.0

The figures given opposite C and C' are the Deflection Coefficients for steel shapes subject to transverse strain for varying spans, under their maximum uniformly distributed safe loads, derived from a fibre strain of 16,000 and 12,500 respectively, the modulus of elasticity being taken at 29,000,000 lbs.

*To find the deflection of any symmetrical shape used as a beam, under its corresponding safe load,** divide the coefficients given in the above tables by the depth of the beam; the result will be the deflection in 64ths of an inch. This applies to such shapes as beams, channels, etc. For those shapes having unsymmetrical sections, such as tees, angles, etc., divide by twice the greatest distance of the neutral axis from the outside fibre.

For a beam supported at both ends and loaded at the centre

* This applies only to the loads at the left of the heavy line in the tables following; the loads to the right of this line being reduced by the rule for stiffness, the deflections obtained by this rule would be excessive.

with one-half the distributed load the deflection will be .8 that obtained by the above table.

EXAMPLE.—Required the deflection of a 10-inch 25-lb. steel beam, 10-foot span, under its maximum distributed load of 13 tons (16,000 lbs. fibre strain). The above table gives 106 as the deflection coefficient; dividing by the depth of the beam (10) we have $\frac{10.6}{64}$ for the deflection at the centre. This is equivalent to .165 in. By formula 1, Chapter XVIII., we find the deflection for the same beam, span, and load $= \frac{26{,}000 \times 1{,}728{,}000 \times .013}{29{,}000{,}000 \times 122.1} = .165$ in., the two results agreeing perfectly. For the same beam and 18-ft. span, with a load of 7.2 tons, we find the deflection by the above table to be $\frac{34.3}{64}$ or .536 in., and by formula 1, .532, or practically the same.

For a concentrated load of 6,500 lbs. at centre of 18-ft. span the deflection would be $.8 \times .53$ or .42 in. As a rule it is not desirable to subject any beam to a load which will produce a deflection at the centre exceeding $\frac{1}{360}$ th of the span, or $\frac{1}{30}$th of an inch per foot of span. A greater deflection is liable to produce cracks in plastered ceilings, and if the beam is exposed the deflection is painful to the eye.

In the tables giving the safe loads for I-beams and channels all of the loads given are within this limit, the loads to the right of the heavy line having been reduced to conform with the rule for stiffness.*

When the deflection is of no particular consequence these loads may be increased to the value obtained by dividing the coefficient, *C*, by the span, but as a general rule, the loads should not exceed those given in the table.

Strut Beams.—It cannot be considered as good engineering to subject a strut to a cross-strain, as any such strain must produce a certain amount of flexure in the strut, which the compressive stress tends to increase.

There are often cases, however, where practical considerations make it desirable to use a strut as a beam also, as in the top chord or principals of trusses. For determining the size of the section in such cases the following method should be used:

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* This rule is as follows:—Multiply the load given immediately to the left of the heavy line by the square of the corresponding span, and divide by the square of the required span; the result will be the required load.

1st. Find the section modulus for the transverse load by formulas $2A$ to $6A$, using 12,000 lbs. as the value of S, and find the area of a section corresponding to the value of R thus found.

2d. Find the section area required by a section of the size found to resist the compression stress by dividing the stress by the value opposite $\frac{l}{r}$ in column II., Table XI., Chapter XIV.

3d. Add the two areas thus found together and use the next larger section having the required area.

Example.—The principal rafter in a truss, $8\frac{1}{2}$ ft. long between joints, supports the end of a purlin at the centre of the span; the weight from the purlin is 2,800 lbs. and the compressive stress is 30,000 lbs. It is desired to use two angles, with long legs vertical and $\frac{1}{2}$ inch apart for the principal. What should be their size?

Ans. By formula ($3A$), $R = \frac{3 \times 2{,}800 \times 8\frac{1}{2}}{12{,}000} = 5.95.$

As two angles will be used, R for each will be 2.98. From the table of properties of angles (p. 304, col. VII.), we find that the angle having a value next above 2.98 is a $5 \times 3 \times 9/16$ inch. The area of this angle is 4.18 in., and from the table on page 317 we find the least value of r_0 for a pair to be about 1.58 (the strut being braced sideways); then $\frac{l}{r} = \frac{102}{1.58} = 64.5$, and p from column II., Table XI., p. 463, $= 10{,}756$ lbs. $30{,}000 \div 10{,}756 = 2.79$ sq. in., or 1.40 in. for each angle. The area of the angle found for the beam was 4.18; adding to this 1.40, we have 5.58 as the required area for each angle, which is found in the $5 \times 3 \times \frac{13}{16}''$ size.

As the area in both steps considerably exceeds that required by the calculation, we need not make further allowance for the weight of the angles.

Tie Beams.—Steel beams subject to both tensile and transverse strain should be calculated in a similar way to that explained above for strut beams. The section necessary to resist the transverse strain should first be found, and then the sectional area necessary to resist the tensile strain, and the two added together.

Example.—One span of a tie beam, 10 ft. between joints, has to support a load at the centre of three tons, and a tensile stress of 84,000 lbs. Two steel channels will be used for the tie. What should be their size and weight?

Ans. A centre load of 3 tons corresponds with a distributed load of 6 tons, or 3 tons for each channel. From the table giving the strength of channels, we find that a light 7-inch channel will be required, the sectional area being 2.85 sq. in. The area required to resist the tensile stress $= \frac{84,000}{14,000} = 6$ sq. in., or 3 in. for each channel, and the total area for each channel should be $2.85+3=5.85$ sq. in. A 7-in. $19\frac{3}{4}$-lb. channel has an area of 5.81 sq. in. which will answer, as we do not use the full strength of the light channel. If a sufficiently heavy section could not be found in the 7-inch channels, we should use the next size, or 8-inch.

Explanation of Tables for the Strength of Steel Beams.

The following tables give the greatest safe loads for all of the standard sections of beams, channels, and angles, and for the Carnegie deck-beams and tees, and also the limits for deflection and buckling of the beams and channels. By following the explanations given below these tables may be used with simple or no computations (other than those required for determining the load they will have to support) for the usual conditions of building construction. For several concentrated loads or for a combination of distributed and concentrated loads it will be necessary to use the methods previously explained under Case 7.

In using any of the following tables allowance should be made for the weight of the beam itself.

I-beams and channels having loads and spans to the *left* of the dotted line, should have the web stiffened.

The loads to the right of heavy line (I-beams and channels) were computed by formula for deflection. "Max. Load" is the greatest distributed load that should be used without stiffening the web by plates or angles. (See page 508.) To find the safe load for any other span than those given in the table, divide the number in column headed C by the given span in feet and decimals of a foot, and the answer will be the safe load for that span, *provided* it is less than the max. load and within the limits of deflection, and is stayed laterally.

To use any of the following tables for CONCENTRATED LOADS, find the equivalent distributed load by multiplying the con-

centrated load by the factor given below, and then use the size of beam having a safe load equal to the load thus found.

For concentrated load at centre, multiply by 2.

For load applied			*one-third*	the span from one end, m'ply by							1.78
"	"	"	*one-fourth*	"	"	"	"	"	"	"	1.5
"	"	"	*one-fifth*	"	"	"	"	"	"	"	1.28
"	"	"	*one-sixth*	"	"	"	"	"	"	"	$1\frac{1}{9}$
"	"	"	*one-seventh*	"	"	"	"	"	"	"	.98
"	"	"	*one-eighth*	"	"	"	"	"	"	"	$\frac{7}{8}$
"	"	"	*one-ninth*	"	"	"	"	"	"	"	.79
"	"	"	*one-tenth*	"	"	"	"	"	"	"	.72

For *two* equal loads applied *one-third* the span from each end, multiply one load by $2\frac{2}{3}$.

For two equal loads applied *one-fourth* the span from each end multiply one load by 2.

For beam fixed at one end, and loaded at the other, multiply by 8.

For beam fixed at one end, and uniformly loaded over entire length, multiply by four.

Examples of Application.

Example 1.—A steel I-beam of 15-foot span has to support a load of 4 tons at a point 5 feet from one support. What should be the size of the beam?

Ans. Five feet is one-third of the span. Multiplying the load by 1.78, we have 7.12 tons. Looking in the following table for the strength of steel I-beams, we find that a 9-inch 25-lb. beam 15-ft. span has a safe load of 7.2 tons, hence this beam will just answer.

Example 2.—A steel I-beam of 15-ft. span supports two equal loads of three tons each, applied 5 feet from each end. What should be the size of the beam?

Ans. Five feet being one-third of the span, multiply one load by $2\frac{2}{3}$, which gives 8 tons as the equivalent distributed load. This will require a 9-inch 30-lb. beam.

[Note.—The same results should be obtained by using the formula 6*A*, page 503.]

Following the tables giving the safe loads for channels is a table computed by the author, giving the strength of small rectangular steel bars. These bars are often used for supporting metal lath in suspended ceilings, and the table will be found useful in determining the size of bar to use for any given span and spacing.

SAFE DISTRIBUTED LOADS IN TONS FOR STANDARD STEEL I-BEAMS.

Including weight of beam:
For fibre strain of 16,000 lbs. and safe deflection.
For live loads, or where subject to impact reduce these loads 20 per cent.
For other methods of loading, and for explanation of *C* and Max. Load, and of zigzag lines, see page 513.

Depth of beam in inches.	Weight per foot in lbs.	*C.*	Max. Load.	Span in feet. 15	16	17	18	19	20	21	22	23	24	25	26	28	30	33	36
24	80	927	37.9	61.8	58.0	54.6	51.5	48.8	46.4	44.2	42.2	40.3	38.6	37.1	35.7	33.1	30.9	28.1	25.8
	85	963	48.5	64.2	60.2	56.6	53.4	50.7	48.1	45.8	43.8	41.9	40.1	38.5	37.0	34.4	32.1	29.2	26.7
	90	995	50.9	66.2	62.2	58.5	55.2	52.4	49.7	47.4	45.2	43.3	41.4	39.8	38.3	35.5	33.1	30.1	27.6
	95	1026	59.7	68.4	64.1	60.3	57.0	54.0	51.3	48.8	46.6	44.6	42.7	41.0	39.5	36.6	34.2	31.1	28.5
	100	1058	71.7	70.4	66.1	62.2	58.7	55.7	52.9	50.4	48.1	46.0	44.0	42.3	40.7	37.8	35.2	32.0	29.3
20	65	623	37.1	41.6	39.0	36.7	34.6	32.8	31.2	29.7	28.3	27.1	26.0	24.9	24.0	22.3	20.8	18.9	15.8
	70	650	49.1	43.2	40.6	38.2	36.0	34.2	32.5	30.9	29.5	28.3	27.0	26.0	25.0	23.2	21.6	19.7	16.5
	75	676	61.5	45.0	45.2	39.8	37.5	35.6	33.8	32.2	30.7	29.4	28.1	27.0	26.0	24.1	22.5	20.5	17.2
	80	782	60.3	52.1	48.9	46.0	43.4	41.1	39.1	37.2	35.5	34.0	32.6	31.3	30.1	27.9	26.0	23.7	19.9
	85	804	73.1	53.6	50.2	47.3	44.6	37.0	40.2	38.3	36.5	35.0	33.5	32.1	30.9	28.7	26.8	24.3	20.4
	90	830	86.8	55.2	51.8	48.8	46.0	43.7	41.5	39.5	37.7	36.1	34.5	33.2	31.9	29.6	27.6	25.1	21.1
	95	857	79.3	57.0	53.5	50.4	47.6	45.1	42.8	40.8	38.9	37.3	35.7	34.3	33.0	30.6	28.5	26.0	21.8
	100	883	92.4	58.8	55.2	52.0	49.0	46.5	44.1	42.0	40.1	38.4	36.8	35.3	34.0	31.5	29.4	26.8	22.4
18	55	471	32.8	31.4	29.4	27.7	26.2	24.8	23.6	22.4	21.4	20.5	19.6	18.8	18.1	16.8	15.7	12.9	10.9
	60	499	45.3	33.2	31.2	29.3	27.7	26.2	24.9	23.7	22.7	21.7	20.8	19.9	19.2	17.8	16.6	13.7	11.5
	65	522	58.1	34.8	32.6	30.6	29.0	27.5	26.1	24.8	23.7	22.7	21.7	20.9	20.1	18.6	17.4	14.4	12.1
	70	546	57.2	36.4	34.1	32.1	30.3	28.7	27.3	26.0	24.8	23.7	22.7	21.8	21.0	19.5	112	15.0	12.6

SAFE DISTRIBUTED LOADS IN TONS FOR STANDARD STEEL I-BEAMS (*continued*).

(See pages 513 and 515.)

Depth of beam in inches.	Weight per foot in lbs.	*C.*	Max. Load.	Span in Feet. 14	15	16	17	18	19	20	21	22	23	24	25	26	28	30
18*	75	606	70.9	43.3	40.4	37.9	35.7	33.7	31.9	30.3	28.9	27.5	26.3	25.2	24.2	23.3	21.6	20.2
	80	630	84.2	45.0	42.0	39.4	37.0	35.0	33.1	31.5	30.0	28.6	27.4	26.2	25.2	24.2	23.3	22.5
	85	690	76.0	49.3	46.0	43.1	40.6	38.3	36.3	34.5	32.8	31.3	30.0	28.7	27.6	26.5	24.6	23.0
	90	713	89.1	50.9	47.5	44.5	41.9	39.6	37.5	35.6	33.9	32.4	31.0	29.7	28.5	27.4	25.4	23.7
15	42	314	23.8	22.4	20.9	19.6	18.5	17.4	16.5	15.7	14.9	14.3	13.6	13.0	12.5	11.6		
	45	324	31.2	23.1	21.6	20.2	19.0	18.0	17.0	16.2	15.4	14.7	14.1	13.5	12.9	12.0		
	50	343	35.1	24.5	22.8	21.4	20.2	19.0	18.0	17.1	16.3	15.6	14.9	14.3	13.7	12.7		
	55	363	48.1	25.9	24.2	22.6	21.3	20.0	19.1	18.1	17.3	16.5	15.8	15.1	14.5	13.4		
	60	433	44.7	30.9	28.8	27.0	25.4	24.0	22.8	21.6	20.6	19.7	18.8	18.0	17.3	16.0	13.8	12.0
	65	452	57.8	32.3	30.1	28.2	26.6	25.0	23.8	22.6	21.5	20.5	19.6	18.8	18.1	16.7	14.4	12.5
	70	471	54.9	33.6	31.4	29.4	27.7	26.1	24.8	23.5	22.4	21.4	20.5	19.6	18.8	17.4	15.0	13.0
	75	491	68.0	35.1	32.7	30.7	28.8	27.2	25.8	24.5	23.4	22.3	21.3	20.4	19.6	18.1	15.6	13.6
	80	565	81.3	40.3	37.7	35.3	32.2	31.4	29.7	28.2	26.9	25.7	24.6	23.5	22.6	20.9	18.0	15.7
	85	581	91.1	41.5	38.7	36.3	34.2	32.8	30.6	29.0	27.6	26.4	25.3	24.6	23.2	21.5	18.5	16.1
	90	601	101.7	42.9	40.0	37.6	35.3	33.4	31.6	30.0	28.6	27.3	26.1	25.0	24.0	22.2	19.1	16.7
	95	620	114.8	44.3	41.3	38.7	36.5	34.4	32.6	31.0	29.5	28.2	27.0	25.8	24.8	22.9	19.8	17.2
	100	640	127.8	45.7	42.7	40.0	37.6	35.5	33.7	32.0	30.5	29.1	27.8	26.6	25.6	23.7	20.4	17.7
12	31.50	191.8	17.8	13.7	12,8	11.9	11.3	10.6	10.1	9.6	8.7	7.9						
	35.00	202.9	26.6	14.5	13.5	12.6	11.9	11.3	10.6	10.1	9.2	8.4						
	40.00	239	26.1	17.0	15.9	14.9	14.0	13.3	12.6	11.9	10.8	9.8	9.0	8.3	7.6			
	45.00	254	39.2	18.1	16.9	15.8	14.9	14.1	13.4	12.7	11.5	10.5	9.6	8.8	8.1			
	50.00	269	40.1	19.2	17.9	16.8	15.8	14.9	14.1	13.4	12.2	11.1	10.1	9.3	8.6			
	55.00	285	40.9	20.3	19.0	17.8	16.8	15.3	15.0	14.2	12.9	11.8	10.9	9.9	9.1			

* Rolled only by the Pencoyd and Passaic Mills.

SAFE DISTRIBUTED LOADS IN TONS FOR STANDARD STEEL I-BEAMS (*continued*).

(See pages 513 and 515.)

Depth of beam in inches.	Weight per foot in lbs.	*C.*	Max. Load.	Span in feet. 8	9	10	11	12	13	14	15	16	17	18	19	20	22
12	60	342.9	54.2	42.8	38.1	34.3	31.1	28.6	26.4	24.5	22.8	21.4	20.1	19.0	18.0	17.1	15.6
	65	358.7	67.2	44.8	39.8	35.8	32.6	29.9	27.6	25.6	23.9	22.4	21.1	19.9	18.9	17.9	16.3
10	25	130.2	13.5	16.2	14.4	13.0	11.8	10.8	10.0	9.3	8.6	8.1	7.2	6.4			
	30	142.1	26.6	17.9	15.9	14.3	13.0	11.9	11.0	10.2	9.5	8.9	7.9	7.1			
	35	156.2	26.2	19.5	17.3	15.6	14.2	13.0	12.0	11.1	10.4	9.7	8.6	7.7			
	40	169.2	39.4	21.1	18.8	16.9	15.4	14.1	13.0	12.1	16.6	10.5	9.7	8.3			
9	21	100.6	10.6	12.5	11.1	10.0	9.1	8.4	7.7	7.2	6.7	5.9	5.2	4.6			
	25	108.9	20.9	13.6	12.1	10.9	9.9	9.0	8.4	7.8	7.2	6.4	5.6	5.0			
	30	120.7	34.1	15.1	13.4	12.0	10.9	10.0	9.3	8.6	8.0	7.0	6.2	5.6			
	35	132.5	46.9	16.5	13.6	13.2	12.0	11.0	10.2	9.4	8.8	7.7	6.9	6.1			
8	18.00	75.8	9.7	9.4	8.4	7.5	6.9	6.3	5.8	5.0	4.3						
	20.50	80.8	16.3	10.1	8.9	8.0	7.3	6.7	6.2	5.3	4.6						
	23.00	86.0	22.9	10.7	9.5	8.6	7.8	7.1	6.6	5.7	4.9						
	25.50	91.2	29.4	11.4	10.1	9.1	8.3	7.6	7.0	6.0	5.2						
7	15.00	55.2	8.6	6.9	6.1	5.5	5.0	4.6	3.9	3.4	2.9						
	17.50	59.7	15.1	7.4	6.6	5.9	5.4	4.9	4.2	3.6	3.2						
	20.00	64.3	21.6	8.0	7.1	6.4	5.8	5.3	4.5	3.9	3.4						
6	12.25	38.7	6.9	4.6	4.3	3.8	3.2	2.7	2.3								
	14.75	42.6	13.5	5.3	4.7	4.2	3.5	2.9	2.5								
	17.25	46.5	19.9	5.8	5.1	4.6	3.8	3.2	2.7								

SAFE DISTRIBUTED LOADS IN TONS FOR STANDARD STEEL I-BEAMS (*continued*).

(See pages 513 and 515.)

Depth of beam in inches.	Weight per foot, lbs.	C.	Max. Load.	Span in feet.								
				4	5	6	6½	7	8	9	10	11
5	9.75	25.80	5.5	6.45	5.16	4.30	3.97	3.68	3.22	2.54	2.06	1.70
	12.25	29.05	12.1	7.26	5.81	4.84	4.47	4.15	3.63	2.87	2.32	1.92
	14.75	32.30	18.4	8.07	6.46	5.38	4.97	4.61	4.03	3.19	2.58	2.13
4	7.50	15.90	4.1	3.97	3.18	2.65	2.44	2.27	1.74	1.37	1.11	
	8.50	16.95	6.7	4.24	3.39	2.82	2.60	2.42	1.85	1.46	1.18	
	9.50	18.00	9.2	4.50	3.60	3.00	2.77	2.57	1.97	1.55	1.26	
	10.50	19.05	11.7	4.76	3.81	3.17	2.93	2.72	2.08	1.64	1.33	
3	5.50	8.80	2.7	2.20	1.76	1.46	1.25	1.08	0.82	0.65	0.53	
	6.50	9.55	5.3	2.39	1.91	1.59	1.35	1.17	0.89	0.71	0.57	
	7.50	10.35	7.8	2.58	2.07	1.72	1.47	1.27	0.97	0.76	0.62	

SAFE DISTRIBUTED LOADS IN TONS FOR CARNEGIE DECK-BEAMS.

(For dead loads only.)

(See explanation, page 513.)

c = load to be added to C for each lb. increase in weight of beam.

Depth of section, ins.	Weight per foot, lbs.	C.	c.	Max. Load.	Maximum fibre strain, 16,000 lbs. per square inch. Span in feet.							
					6	8	9	10	12	14	16	18
11½	37.00	163.2	....	39.2	27.21	20.40	18.14	16.32	13.60	11.66	10.20	9.07
11½	32.20	147.4	3.04	25.7	24.56	18.42	16.37	14.74	12.28	10.53	9.21	8.19
10	35.70	137.1	....	43.4	22.84	17.13	15.23	13.71	11.42	9.79	8.57	7.61
10	27.23	113.1	2.45	20.8	18.84	14.13	12.56	11.31	9.42	8.08	7.07	6.28
9	30.00	104.3	....	35.4	17.37	13.03	11.59	10.43	8.69	7.45	6.52	5.79
9	26.00	94.5	2.25	24.7	15.76	11.81	10.51	9.45	7.88	6.75	5.91	5.25
8	24.48	75.1	....	25.3	12.51	9.39	8.35	7.51	6.25	5.36	4.69	3.70
8	20.15	64.9	2.00	13.7	10.82	8.11	7.21	6.49	5.41	4.64	4.06	3.20
7	23.46	62.3	....	27.2	10.38	7.79	6.92	6.23	5.19	4.45	3.40	2.69
7	18.11	51.5	1.7	13.0	8.58	6.44	5.72	5.15	4.29	3.68	2.81	2.22
6	17.16	38.4	....	18.2	6.40	4.80	4.27	3.84	3.20	2.35	1.80	1.42
6	14.10	32.5	1.5	10.3	5.42	4.07	3.62	3.25	2.71	1.99	1.52	1.20

SAFE DISTRIBUTED LOADS IN TONS FOR STANDARD STEEL CHANNELS.

Including weight of channel. Calculated for fibre strain of 16,000 lbs. and safe deflection.

For live loads, or where subject to impact reduce these loads 20 per cent.

For other methods of loading, and for explanation of *C* and Max. Load, and of zigzag lines, see page 513.

Depth, inches.	Weight per foot. lbs.	*C.*	Maximum load.	Span in feet.															
				10	12	14	15	16	17	18	19	20	21	22	23	24	26	28	30
15	33.00	222.2	22.5	22.2	18.5	15.8	14.8	13.9	13.0	12.3	11.7	11.1	10.5	10.1	9.6	9.25	8.54	7.93	6.91
	35.00	227.5	27.4	22.7	18.9	16.2	15.1	14.2	13.4	12.6	12.0	11.3	10.8	10.3	9.9	9.45	8.75	8.12	7.08
	40.00	247.1	40.0	24.7	20.6	17.6	16.4	15.4	14.5	13.7	13.0	12.3	11.7	11.2	10.7	10.30	9.50	8.82	7.68
	45.00	266.7	53.1	26.6	22.2	19.0	17.7	16.6	15.7	14.8	14.0	13.3	12.7	12.1	11.6	11.11	10.22	9.52	8.30
	50.00	286.3	54.5	28.6	23.8	20.4	19.0	17.9	16.8	15.9	15.0	14.3	13.6	13.0	12.4	11.93	11.01	10.22	8.90
	55.00	305.9	67.7	30.6	25.5	21.8	20.3	19.1	17.9	17.0	16.1	15.3	14.5	13.9	13.3	12.74	11.76	10.92	9.52
12	20.50	113.9	11.7	11.4	9.5	8.1	7.5	7.1	6.7	6.3	6.0	5.7	5.42	5.18	4.73	4.35	3.70	3.19	
	25.00	128.0	22.5	12.8	10.6	9.1	8.5	8.0	7.5	7.1	6.7	6.4	6.09	5.82	5.32	4.88	4.16	3.59	
	30.00	143.7	35.7	14.3	11.9	10.2	9.5	8.9	8.4	7.9	7.5	7.1	6.84	6.53	5.98	5.49	4.67	4.03	
	35.00	159.4	34.7	15.9	13.3	11.4	10.6	9.9	9.4	8.8	8.3	7.9	7.59	7.24	6.63	6.08	5.18	4.47	
	40.00	175.1	47.8	17.5	14.6	12.5	11.6	10.9	10.3	9.7	9.2	8.7	8.34	7.96	7.28	6.69	5.70	4.91	
10	15.00	71.3	8.2	7.1	5.9	5.1	4.75	4.45	4.19	3.96	3.55	3.21	2.91	2.65					
	20.00	84.0	20.6	8.4	7.0	6.0	5.60	5.25	4.94	4.66	4.18	3.78	3.43	3.12					
	25.00	97.0	27.4	9.7	8.0	6.9	6.46	6.06	5.70	5.29	4.83	4.36	3.96	3.60					
	30.00	110.1	40.5	11.0	9.1	7.8	7.34	6.88	6.47	6.10	5.49	4.95	4.49	4.09					
	35.00	123.2	53.4	12.3	10.2	8.8	8.21	7.70	7.24	6.83	6.14	5.54	5.03	4.58					
9	13.25	56.1	7.8	5.6	4.6	4.00	3.74	3.50	3.10	2.77	2.48	2.24							
	15.00	60.2	12.1	6.0	5.0	4.30	4.01	3.76	3.33	2.97	2.67	2.41							
	20.00	72.0	19.8	7.2	6.0	5.14	4.80	4.50	3.98	3.55	3.19	2.88							
	25.00	83.8	33.1	8.4	6.9	5.98	5.58	5.23	4.64	4.14	3.71	3.35							

SAFE DISTRIBUTED LOADS IN TONS FOR STANDARD STEEL CHANNELS (*continued*).

Depth, inches.	Weight per foot, lbs.	C.	Max. Load.	Span in feet. 3	4	5	6	7	8	9	10	11	12	13	14	15	16
8	11.25	43.05	6.8	14.3	10.7	8.6	7.1	6.1	5.4	4.8	4.30	3.91	3.59	3.31	3.07	2.86	2.52
	13.75	48.00	13.2	16.0	12.0	9.6	8.0	6.9	6.0	5.3	4.80	4.36	4.00	3.69	3.42	3.20	2.81
	16.25	53.20	15.4	17.7	13.3	10.6	8.9	7.6	6.6	5.9	5.32	4.83	4.43	4.09	3.80	3.54	3.14
	18.75	58.45	22.0	19.4	14.6	11.7	9.7	8.3	7.3	6.5	5.84	5.31	4.87	4.50	4.17	3.89	3.42
	21.25	63.70	28.5	21.2	15.9	12.7	10.6	9.1	7.9	7.1	6.37	5.80	5.30	4.90	4.55	4.24	3.73
7	9.75	33.40	6.6	11.1	8.3	6.7	5.56	4.77	4.17	3.71	3.34	3.03	2.78	2.37	2.04	1.78	1.56
	12.25	36.85	13.2	12.3	9.2	7.3	6.14	5.26	4.60	4.09	3.68	3.35	3.07	2.61	2.25	1.96	1.73
	14.75	41.40	16.0	13.4	10.3	8.3	6.90	5.91	5.17	4.60	4.14	3.76	3.45	2.94	2.53	2.20	1.94
	17.25	46.00	22.6	15.3	11.5	9.2	7.66	6.57	5.75	5.11	4.60	4.18	3.83	3.27	2.81	2.45	2.15
	19.75	50.55	28.9	16.8	12.6	10.1	8.42	7.22	6.32	5.62	5.05	4.60	4.21	3.59	3.09	2.69	2.37
6	8.00	23.10	5.4	7.7	5.8	4.6	3.85	3.30	2.88	2.57	2.31	1.91	1.60	1.37			
	10.50	26.90	10.0	8.9	6.7	5.4	4.48	3.84	3.36	2.99	2.69	2.22	1.86	1.59			
	13.00	30.80	16.5	10.2	7.7	6.1	5.13	4.40	3.85	3.42	3.08	2.54	2.14	1.82			
	15.50	34.75	22.9	11.6	8.7	6.9	5.79	4.96	4.34	3.86	3.47	2.87	2.41	2.07			
5	6.50	15.80	4.6	5.2	3.9	3.16	2.63	2.25	1.97	1.56	1.26	1.04					
	9.00	18.95	11.1	6.3	4.7	3.79	3.16	2.70	2.37	1.87	1.51	1.25					
	11.50	22.20	10.7	7.4	5.5	4.44	3.70	3.17	2.77	2.19	1.77	1.47					
4	5.25	10.10	4.1	3.36	2.52	2.02	1.68	1.44	1.10	.87							
	6.25	11.15	6.7	3.71	2.78	2.23	1.86	1.59	1.22	.96							
	7.25	12.20	9.2	4.06	3.05	2.44	2.03	1.74	1.33	1.05							
3	4.00	5.80	3.0	1.93	1.45	1.16	.80	.59									
	5.00	6.55	5.5	2.18	1.64	1.31	.91	.67									
	6.00	7.35	8.0	2.45	1.84	1.47	1.02	.75									

SAFE DISTRIBUTED LOADS IN TONS FOR STANDARD STEEL CHANNELS—SET FLATWAYS

or with the load acting at right angles to the plane of the web.

(Computed by the Pencoyd Iron Works.)
(Fibre Stress 16,000 lbs. per Square Inch.)

Size in ins.	Wt. in lbs. per foot.	Length of span in feet.									
		4	5	6	7	8	9	10	11	12	13
		Safe load in net tons.									
15	33.0	4.14	3.31	2.76	2.36	2.07	1.84	1.66	1.50	1.38	1.27
15	35.0	4.28	3.42	2.85	2.45	2.14	1.90	1.71	1.56	1.43	1.32
15	40.0	4.56	3.65	3.04	2.61	2.28	2.03	1.82	1.66	1.52	1.40
15	45.0	4.83	3.86	3.22	2.76	2.41	2.15	1.93	1.76	1.61	1.49
15	50.0	6.92	5.53	4.61	3.95	3.46	3.07	2.77	2.51	2.31	2.13
15	55.0	7.30	5.84	4.87	4.17	3.65	3.25	2.92	2.66	2.43	2.25
12	20.5	2.32	1.86	1.55	1.33	1.16	1.03	0.93	0.84	0.77	0.71
12	25.0	2.53	2.03	1.69	1.45	1.27	1.13	1.01	0.92	0.84	0.78
12	30.0	2.77	2.22	1.85	1.58	1.39	1.23	1.11	1.01	0.92	0.85
12	35.0	4.76	3.81	3.17	2.72	2.38	2.12	1.90	1.73	1.59	1.46
12	40.0	5.12	4.10	3.41	2.93	2.56	2.28	2.05	1.86	1.71	1.58
10	15.0	1.55	1.24	1.03	0.88	0.77	0.69	0.62	0.56	0.52	0.48
10	20.0	1.77	1.42	1.18	1.01	0.89	0.79	0.71	0.64	0.59	0.54
10	25.0	2.66	2.13	1.77	1.52	1.33	1.18	1.06	0.97	0.89	0.82
10	30.0	2.95	2.36	1.97	1.69	1.47	1.31	1.18	1.07	0.98	0.91
10	35.0	3.26	2.61	2.17	1.86	1.63	1.45	1.30	1.19	1.09	1.00
9	13.25	1.29	1.03	0.86	0.74	0.65	0.57	0.52	0.47	0.43	0.40
9	15.00	1.37	1.10	0.92	0.79	0.69	0.61	0.55	0.50	0.46	0.42
9	20.00	2.08	1.67	1.39	1.19	1.04	0.92	0.83	0.76	0.69	0.64
9	25.00	2.37	1.89	1.58	1.35	1.18	1.05	0.95	0.86	0.79	0.73
8	11.25	1.04	0.83	0.70	0.60	0.52	0.46	0.42	0.38	0.35	0.32
8	13.75	1.16	0.92	0.77	0.66	0.58	0.51	0.46	0.42	0.39	0.36
8	16.25	1.64	1.31	1.09	0.94	0.82	0.73	0.66	0.59	0.55	0.50
8	18.75	1.77	1.42	1.18	1.01	0.89	0.79	0.71	0.64	0.59	0.55
8	21.25	1.90	1.52	1.27	1.09	0.95	0.85	0.76	0.69	0.63	0.59
7	9.75	0.84	0.67	0.56	0.48	0.42	0.37	0.34	0.31	0.28	0.26
7	12.25	0.95	0.76	0.63	0.54	0.47	0.42	0.38	0.34	0.32	0.29
7	14.75	1.37	1.10	0.91	0.78	0.69	0.61	0.55	0.50	0.46	0.42
7	17.25	1.51	1.20	1.00	0.86	0.75	0.67	0.60	0.55	0.50	0.46
7	19.75	1.64	1.31	1.10	0.94	0.82	0.73	0.66	0.60	0.55	0.50
6	8.00	0.65	0.52	0.44	0.37	0.33	0.29	0.26	0.24	0.22	0.20
6	10.50	0.91	0.73	0.61	0.52	0.45	0.41	0.36	0.33	0.30	0.28
6	13.00	1.04	0.83	0.69	0.59	0.52	0.46	0.41	0.38	0.35	0.32
6	15.50	1.16	0.93	0.78	0.66	0.58	0.52	0.46	0.42	0.39	0.36
5	6.50	0.50	0.40	0.33	0.28	0.25	0.22	0.20	0.18	0.16	0.15
5	9.00	0.61	0.49	0.41	0.35	0.30	0.27	0.25	0.22	0.20	0.19
5	11.50	0.72	0.57	0.48	0.41	0.36	0.32	0.29	0.26	0.24	0.22

SAFE DISTRIBUTED LOADS IN POUNDS FOR SMALL STEEL CHANNELS, OR GROOVED STEEL.

(Computed for fibre strain of 16,000 lbs.)

(For dimensions of sections see page 300.)

Section No.	D'pth in ins.	Wt. per foot, lbs.	C lbs.	Span in feet. 2	2.5	3	3.5	4	4.5	5	6
1	2¼	3.80	7570	3785	3028	2523	2163	1892	1682	1514	1261
2	2	2.90	5120	2560	2048	1706	1463	1280	1138	1024	853
3	2	3.60	5760	2880	2304	1920	1643	1440	1280	1152	960
4	2	3.60	6240	3120	2496	2080	1783	1560	1386	1248	1040
5	2	2.60	4512	2256	1804	1504	1289	1128	1000	902	752
6	2	2.00	2836	1418	1134	945	810	709	630	567	472
7	1¾	1.13	1815	907	726	605	518	454	403	363	302
8	1½	1.32	1536	768	614	512	439	384	341	307	256
9	1½	1.46	1736	868	694	578	496	434	386	347	289
10	1¼	0.94	950	475	380	316	271	237	211	190	
11	1⅛	1.12	939	469	375	313	268	234	208	188	
12	1⅛	1.00	874	437	350	291	250	218	194	175	
13	1	0.83	672	336	268	224	192	168			
14	1	0.68	532	266	212	177	152	133			
15	⅞	0.67	448	224	180	149	128	112			
16	⅞	0.69	458	229	183	152	130				
17	¾	0.53	266	133	106	88					

SAFE DISTRIBUTED LOADS IN POUNDS FOR RECTANGULAR STEEL BARS, ON EDGE, USED AS BEAMS.

(Computed for a fibre stress of 16,200 lbs.)

Depth in ins.	Thickness.	Span in Feet.							
		2	2½	3	3½	4	4½	5	5½
1	¼	225	180	150					
1	5/16	281	225	187					
1	⅜	337	270	225					
1¼	¼	350	280	234	200	175			
1¼	5/16	438	350	292	250	219			
1¼	⅜	526	420	350	300	262			
1½	¼	506	405	338	289	253	225		
1½	5/16	632	506	422	361	316	281		
1½	⅜	759	607	506	433	379	337		
1¾	¼	689	551	459	393	344	306	275	
1¾	5/16	861	688	573	491	430	382	343	
1¾	⅜	1033	826	688	589	516	459	412	
2	5/16	1125	900	750	642	562	500	450	409
2	⅜	1350	1080	900	781	675	600	540	490
2	7/16	1575	1260	1050	914	787	700	630	572
2¼	5/16	1423	1138	948	813	711	632	569	517
2¼	⅜	1708	1366	1138	976	853	759	683	621
2¼	7/16	1993	1594	1328	1139	996	885	797	724
2½	5/16	1757	1406	1171	1004	878	781	703	639
2½	⅜	2109	1687	1406	1205	1054	937	843	747
2½	7/16	2460	1968	1540	1406	1230	1093	984	855
3	⅜	3037	2430	2025	1736	1518	1350	1215	1104
3	7/16	3543	2835	2362	2025	1771	1575	1417	1288
3	½	4050	3240	2700	2314	2025	1800	1620	1472

SAFE DISTRIBUTED LOADS IN TONS FOR COMMON SIZES OF STEEL ANGLES.

Computed for fibre stress of 16,000 lbs. For permanent and live loads reduce 20 per cent. See further explanation on page 513.

ANGLES WITH EQUAL LEGS.

Size of angle.	C.	Span in feet.			
		2	3	4	5
8 ×8 ×1⅛	93.49	46.74	31.16	23.37	18.70
8 ×8 × ½	44.64	22.32	14.88	11.16	8.93
6 ×6 ×1	45.72	22.86	15.24	11.43	9.14
6 ×6 × ⅜	18.82	9.41	6.27	4.70	3.76
5 ×5 ×1	30.91	15.45	10.30	7.73	6.18
5 ×5 × ⅜	12.91	6.45	4.30	3.23	2.58
4 ×4 ×13/16	16.05	8.03	5.35	4.01	3.21
4 ×4 ×5/16	6.88	3.44	2.29	1.72	1.38
3½×3½×13/16	12.00	6.00	4.00	3.00	2.40
3½×3½×5/16	5.20	2.60	1.73	1.30	1.04
3 ×3 × ⅝	6.93	3.47	2.31	1.73	1.39
3 ×3 × ¼	3.09	1.55	1.03	0.77	0.62
2¾×2¾× ½	4.75	2.37	1.58	1.19	0.95
2¾×2¾× ¼	2.56	1.28	0.85	0.64	0.51
2½×2½× ½	3.89	1.95	1.29	0.97	0.78
2½×2½×3/16	1.61	0.81	0.54	0.40	0.32
2¼×2¼× ½	3.09	1.55	1.03	0.77	0.62
2¼×2¼×3/16	1.30	0.65	0.43	0.32	0.26
2 ×2 ×7/16	2.13	1.07	0.71	0.53	0.43
2 ×2 ×3/16	1.01	0.51	0.34	0.25	0.20
1¾×1¾×7/16	1.60	0.80	0.53	0.40	0.32
1¾×1¾×3/16	0.75	0.37	0.25	0.19	0.15
1½×1½× ⅜	1.01	0.51	0.34	0.25	0.20
1½×1½× ⅛	0.38	0.19	0.13	0.096	0.077
1¼×1¼×5/16	0.58	0.29	0.19	0.150	0.120
1¼×1¼× ⅛	0.26	0.13	0.087	0.065	0.052
1 ×1 × ¼	0.30	0.15	0.100	0.075	0.060
1 ×1 × ⅛	0.17	0.083	0.055	0.041	0.033
⅞× ⅞×3/16	0.18	0.088	0.059	0.044	
⅞× ⅞× ⅛	0.12	0.061	0.041	0.031	
¾× ¾ ×3/16	0.13	0.064	0.043	0.032	
¾× ¾× ⅛	0.091	0.045	0.030	0.023	

SAFE DISTRIBUTED LOADS IN TONS FOR COMMON SIZES OF STEEL ANGLES (*continued*).

Computed for fibre stress of 16,000 lbs. For permanent and live loads reduce 20 per cent. See further explanation on page 513.

ANGLES WITH EQUAL LEGS.

Size of angle.	Span in feet.				
	6	7	8	9	10
8 ×8 ×1⅛	15.58	13.36	11.69	10.39	9.35
8 ×8 × ½	7.44	6.38	5.58	4.96	4.46
6 ×6 ×1	7.62	6.53	5.72	5.08	4.57
6 ×6 × ⅜	3.14	2.69	2.35	2.09	1.88
5 ×5 ×1	5.15	4.42	3.86	3.43	3.09
5 ×5 × ⅜	2.15	1.84	1.61	1.43	1.29
4 ×4 ×13/16	2.68	2.29	2.01	1.78	1.61
4 ×4 × 5/16	1.15	0.98	0.86	0.76	0.69
3½×3½×13/16	2.00	1.71	1.50	1.33	1.20
3½×3½×5/16	0.87	0.74	0.65	0.58	0.52
3 ×3 × ⅝	1.16	0.99	0.87	0.77	0.69
3 ×3 × ¼	0.52	0.44	0.39	0.34	0.31
2¾×2¾× ½	0.79	0.68	0.59	0.53	0.47
2¾×2¾× ¼	0.43	0.37	0.32	0.28	0.26
2½×2½× ½	0.65	0.56	0.49	0.43	0.39
2½×2½×3/16	0.27	0.23	0.20	0.18	0.16
2¼×2¼× ½	0.52	0.44	0.39	0.34	0.31
2¼×2¼×3/16	0.22	0.19	0.16	0.14	0.13
2 ×2 ×7/16	0.36	0.30	0.27	0.24	0.21
2 ×2 ×3/16	0.17	0.14	0.13	0.11	0.10
1¾×1¾×7/16	0.27	0.23	0.20	0.18	0.16
1¾×1¾×3/16	0.12	0.11	0.093	0.083	0.075
1½×1½× ⅜	0.17	0.14	0.130	0.110	
1½×1½× ⅛	0.064	0.055	0.048	0.043	
1¼×1¼×5/16	0.097	0.083	0.073	0.065	
1¼×1¼× ⅛	0.044	0.037	0.033	0.029	
1 ×1 × ¼	0.050				
1 ×1 × ⅛	0.028				

SAFE DISTRIBUTED LOADS IN TONS FOR COMMON SIZES OF STEEL ANGLES.

Computed for fibre stress of 16,000 lbs. For permanent and live loads reduce 20 per cent. See further explanation on page 513.

ANGLES WITH UNEQUAL LEGS.—LONG LEG VERTICAL.

Size of angle.	*C.*	Span in feet.								
		2	3	4	5	6	7	8	9	10
7 ×3½×1	56.43	28.21	18.81	14.11	11.29	9.40	8.06	7.05	6.27	5.64
7 ×3½× 7/16	26.72	13.36	8.91	6.68	5.34	4.45	3.82	3.34	2.97	2.67
6 ×4 ×1	42.77	21.39	14.26	10.69	8.55	7.13	6.11	5.35	4.75	4.28
6 ×4 × 3/8	17.71	8.85	5.90	4.43	3.54	2.95	2.53	2.21	1.97	1.77
6 ×3½×1	41.76	20.88	13.92	10.44	8.35	6.96	5.97	5.22	4.64	4.18
6 ×3½× 3/8	17.33	8.67	5.78	4.33	3.47	2.89	2.48	2.17	1.93	1.73
5 ×4 × 7/8	26.61	13.31	8.87	6.65	5.32	4.44	3.80	3.33	2.96	2.66
5 ×4 × 3/8	12.48	6.24	4.16	3.12	2.50	2.08	1.78	1.56	1.39	1.25
5 ×3½× 7/8	26.03	13.01	8.68	6.51	5.21	4.34	3.72	3.25	2.89	2.60
5 ×3½× 5/16	10.35	5.18	3.45	2.59	2.07	1.73	1.48	1.29	1.15	1.04
5 ×3 × 13/16	23.73	11.87	7.91	5.93	4.75	3.96	3.39	2.97	2.64	2.37
5 ×3 × 5/16	10.08	5.04	3.36	2.52	2.02	1.68	1.44	1.26	1.12	1.01
4½×3 × 13/16	19.31	9.65	6.44	4.83	3.86	3.22	2.76	2.41	2.15	1.93
4½×3 × 5/16	8.21	4.11	2.74	2.05	1.64	1.37	1.17	1.03	0.91	0.82
4 ×3½× 13/16	15.57	7.79	5.19	3.89	3.11	2.60	2.22	1.95	1.73	1.56
4 ×3½× 5/16	6.72	3.36	2.24	1.68	1.34	1.12	0.96	0.84	0.75	0.67
4 ×3 × 13/16	15.31	7.65	5.10	3.83	3.06	2.55	2.19	1.91	1.70	1.53
4 ×3 × 5/16	6.56	3.28	2.19	1.64	1.31	1.10	0.94	0.82	0.73	0.66
3½×3 × 13/16	11.73	5.87	3.91	2.93	2.35	1.96	1.68	1.47	1.30	1.17
3½×3 × 5/16	5.12	2.56	1.71	1.28	1.02	0.85	0.73	0.64	0.57	0.51
3½×2½× 11/16	9.87	4.93	3.29	2.47	1.97	1.64	1.41	1.23	1.10	0.99
3½×2½× ¼	4.00	2.00	1.33	1.00	0.80	0.67	0.57	0.50	0.44	0.40
3¼×2 × 9/16	6.93	3.47	2.31	1.73	1.39	1.16	0.99	0.87	0.77	0.69
3¼×2 × ¼	3.36	1.68	1.12	0.84	0.67	0.56	0.48	0.42	0.37	0.34
3 ×2½× 9/16	6.13	3.07	2.04	1.53	1.23	1.02	0.88	0.77	0.68	0.61
3 ×2½× ¼	2.99	1.50	1.00	0.75	0.60	0.50	0.43	0.37	0.33	0.30
3 ×2 × ½	5.33	2.67	1.78	1.33	1.07	0.89	0.76	0.67	0.59	0.53
3 ×2 × ¼	2.88	1.44	0.96	0.72	0.58	0.48	0.41	0.36	0.32	0.29
2½×2 × ½	3.73	1.87	1.24	0.93	0.75	0.62	0.53	0.47	0.41	0.37
2½×2 × 3/16	1.55	0.77	0.52	0.39	0.31	0.26	0.22	0.19	0.17	0.16
2¼×1½× ½	3.15	1.57	0.05	0.79	0.63	0.52	0.45	0.39	0.35	0.32
2¼×1½× 3/16	1.23	0.61	0.41	0.31	0.25	0.21	0.18	0.15	0.14	0.12
2 ×1⅜× ¼	1.23	0.61	0.41	0.31	0.25	0.21	0.18	0.15	0.14	0.12
2 ×1⅜× 3/16	0.96	0.48	0.32	0.24	0.19	0.16	0.14	0.12	0.11	0.10
1⅜×1 × ¼	0.48	0.24	0.16	0.12	0.10	0.08	0.07	0.06	0.05	0.05
1⅜×1 × ⅛	0.32	0.16	0.11	0.08	0.06	0.05	0.05	0.04	0.04	0.03

SAFE DISTRIBUTED LOADS IN TONS FOR COMMON SIZES OF STEEL ANGLES.

Computed for fibre stress of 16,000 lbs. For permanent and live loads reduce 20 per cent. See further explanation on page 513.

ANGLES WITH UNEQUAL LEGS.—SHORT LEG VERTICAL.

Size of angle. Inches.	C.	Span in feet.								
		2	3	4	5	6	7	8	9	10
7 ×3½×1	15.79	7.89	5.26	3.95	3.16	2.63	2.26	1.97	1.75	1.58
7 ×3½×7/16	7.84	3.92	2.61	1.96	1.57	1.31	1.12	0.98	0.87	0.78
6 ×4 ×1	20.21	10.11	6.74	5.05	4.04	3.37	2.89	2.53	2.25	2.02
6 ×4 × 3/8	8.53	4.27	2.84	2.13	1.71	1.42	1.22	1.07	0.95	0.85
6 ×3½×1	15.47	7.74	5.16	3.87	3.09	2.58	2.21	1.93	1.72	1.55
6 ×3½× 3/8	6.56	3.28	2.19	1.64	1.31	1.09	0.94	0.82	0.73	0.66
5 ×4 × 7/8	17.65	8.83	5.88	4.41	3.53	2.94	2.52	2.21	1.96	1.77
5 ×4 × 3/8	8.37	4.19	2.79	2.09	1.67	1.40	1.20	1.05	0.93	0.84
5 ×3½× 7/8	13.44	6.72	4.48	3.36	2.69	2.24	1.92	1.68	1.49	1.34
5 ×3½× 5/16	5.44	2.72	1.81	1.36	1.09	0.91	0.78	0.68	0.60	0.54
5 ×3 × 13/16	9.28	4.64	3.09	2.32	1.86	1.55	1.33	1.16	1.03	0.93
5 ×3 × 5/16	4.00	2.00	1.33	1.00	0.80	0.67	0.57	0.50	0.44	0.40
4½×3 × 13/16	9.12	4.56	3.04	2.28	1.82	1.52	1.30	1.14	1.01	0.91
4½×3× 5/16	4.05	2.03	1.35	1.01	0.81	0.68	0.58	0.51	0.45	0.41
4 ×3½× 13/16	12.27	6.13	4.09	3.07	2.45	2.05	1.75	1.53	1.36	1.23
4 ×3½× 5/16	5.39	2.69	1.80	1.35	1.08	0.90	0.77	0.67	0.60	0.54
4 ×3 × 13/16	8.96	4.48	2.99	2.24	1.79	1.49	1.28	1.12	1.00	0.90
4 ×3 × 5/16	3.95	1.97	1.32	0.99	0.79	0.66	0.56	0.49	0.44	0.39
3½×3 × 13/16	8.80	4.40	2.93	2.20	1.76	1.47	1.26	1.10	0.98	0.88
3½×3 × 5/16	3.84	1.92	1.28	0.96	0.77	0.64	0.55	0.48	0.43	0.38
3½×2½× 11/16	5.28	2.64	1.76	1.32	1.06	0.88	0.75	0.66	0.59	0.53
3½×2½× ¼	2.19	1.09	0.73	0.55	0.44	0.36	0.31	0.27	0.24	0.22
3¼×2 × 9/16	2.83	1.41	0.94	0.71	0.57	0.47	0.40	0.35	0.31	0.28
3¼×2 × ¼	1.39	0.69	0.46	0.35	0.28	0.23	0.20	0.17	0.15	0.14
3 ×2½× 9/16	4.37	2.19	1.46	1.09	0.87	0.73	0.62	0.55	0.49	0.44
3 ×2½× ¼	2.13	1.07	0.71	0.53	0.43	0.36	0.30	0.27	0.24	0.21
3 ×2 × ½	2.51	1.25	0.84	0.63	0.50	0.42	0.36	0.31	0.28	0.25
3 ×2 × ¼	1.33	0.67	0.44	0.33	0.27	0.22	0.19	0.17	0.15	0.13
2½×2 × ½	2.45	1.23	0.82	0.61	0.49	0.41	0.35	0.31	0.27	0.25
2½×2 × 3/16	1.07	0.53	0.36	0.27	0.21	0.18	0.15	0.13	0.12	0.11
2¼×1½× ½	1.39	0.69	0.46	0.35	0.28	0.23	0.20	0.17		
2¼×1½×	0.59	0.29	0.20	0.15	0.12	0.10				
2 ×1⅜× ¼	0.64	0.32	0.21	0.16	0.13					
2 ×1⅜× 3/16	0.48	0.24	0.16	0.12						
1⅜×1 × ¼	0.27	0.13	0.09							
1⅜×1 × ⅛	0.16	0.08	0.05							

COEFFICIENT OF STRENGTH IN TONS FOR ALL SIZES AND THICKNESSES OF STEEL ANGLES.

Computed for fibre stress of 16,000 lbs.

For permanent or live loads reduce 20 per cent.

To find safe *distributed* load for any span, divide *C* by span in feet: result will be load in tons; or, to find size of angle for any *distributed* load and span, multiply the load by the span, and select an angle having a value for *C* equal or larger than the product.

For load at centre of span multiply twice the load by the span.

For further explanation see page 513.

ANGLES WITH EQUAL LEGS.

Size in inches.	Thickness of metal.	*C*.	Size in inches.	Thickness of metal	*C*.	Size in inches.	Thickness of metal.	*C*.
8×8	$1\frac{1}{8}$	93.49	5×5	$\frac{13}{16}$	25.87	$3\frac{1}{2}\times3\frac{1}{2}$	$\frac{13}{16}$	12.00
	$1\frac{1}{16}$	88.91		$\frac{3}{4}$	24.16		$\frac{3}{4}$	11.25
	1	84.26		$1\frac{11}{16}$	22.40		$\frac{11}{16}$	10.45
	$\frac{15}{16}$	79.52		$\frac{5}{8}$	20.58		$\frac{5}{8}$	9.65
	$\frac{7}{8}$	74.72		$\frac{9}{16}$	18.72		$\frac{9}{16}$	8.80
	$\frac{13}{16}$	70.92		$\frac{1}{2}$	16.80		$\frac{1}{2}$	7.95
	$\frac{3}{4}$	64.96		$\frac{7}{16}$	14.88		$\frac{7}{16}$	7.04
	$\frac{11}{16}$	60.00		$\frac{3}{8}$	12.91		$\frac{3}{8}$	6.13
	$\frac{5}{8}$	54.93					$\frac{5}{16}$	5.22
	$\frac{9}{16}$	49.81	$4\frac{1}{2}\times4\frac{1}{2}$	$\frac{5}{8}$	16.48			
	$\frac{1}{2}$	44.64		$\frac{9}{16}$	14.98	$3\frac{1}{4}\times3\frac{1}{4}$	$\frac{3}{4}$	7.25
				$\frac{1}{2}$	13.49		$\frac{3}{8}$	5.28
6×6	1	45.71		$\frac{7}{16}$	11.95			
	$\frac{15}{16}$	43.25		$\frac{3}{8}$	10.40	3×3	$\frac{5}{8}$	6.93
	$\frac{7}{8}$	40.75		$\frac{5}{16}$	8.75		$\frac{9}{16}$	6.35
	$\frac{13}{16}$	37.18					$\frac{1}{2}$	5.71
	$\frac{3}{4}$	35.52	4×4	$\frac{13}{16}$	16.05		$\frac{7}{16}$	5.07
	$\frac{11}{16}$	32.91		$\frac{3}{4}$	15.18		$\frac{3}{8}$	4.43
	$\frac{5}{8}$	30.19		$\frac{11}{16}$	13.92		$\frac{5}{16}$	3.78
	$\frac{9}{16}$	30.75		$\frac{5}{8}$	12.80		$\frac{1}{4}$	3.09
	$\frac{1}{2}$	24.95		$\frac{9}{16}$	11.68			
	$\frac{7}{16}$	21.71		$\frac{1}{2}$	10.51	$2\frac{3}{4}\times2\frac{3}{4}$	$\frac{1}{2}$	4.75
	$\frac{3}{8}$	18.82		$\frac{7}{16}$	9.33		$\frac{7}{16}$	4.21
				$\frac{3}{8}$	8.11		$\frac{3}{8}$	3.68
5×5	1	30.93		$\frac{5}{16}$	6.88		$\frac{5}{16}$	3.15
	$\frac{15}{16}$	29.28					$\frac{1}{4}$	2.56
	$\frac{7}{8}$	27.57						

For Weight and Properties of these angles, see Tables Chapter XX.

COEFFICIENT OF STRENGTH IN TONS FOR ALL SIZES AND THICKNESSES OF STEEL ANGLES (*continued*).

Reduce 20 per cent. for permanent and live loads.

See explanation on opposite page.

ANGLES WITH EQUAL LEGS.

Size in inches.	Thickness of metal.	C.	Size in inches.	Thickness of metal.	C.	Size in inches.	Thickness of metal.	C.
$2\frac{1}{2}\times2\frac{1}{2}$	1/2	3.89	2×2	7/16	2.13	$1\frac{1}{2}\times1\frac{1}{2}$	3/16	0.555
	7/16	3.47		3/8	1.87		1/8	.373
	3/8	3.04		5/16	1.60	$1\frac{1}{4}\times1\frac{1}{4}$	5/16	.581
	5/16	2.56		1/4	1.33		1/4	.485
	1/4	2.13		3/16	1.01		3/16	.378
	3/16	1.60	$1\frac{3}{4}\times1\frac{3}{4}$	7/16	1.60		1/8	.261
$2\frac{1}{4}\times2\frac{1}{4}$	1/2	3.09		3/8	1.39	1×1	1/4	.298
	7/16	2.77		5/16	1.23		3/16	.234
	3/8	2.40		1/4	1.01		1/8	.165
	5/16	2.08		3/16	0.75	$\frac{7}{8}\times\frac{7}{8}$	3/16	.176
	1/4	1.71	$1\frac{1}{2}\times1\frac{1}{2}$	3/8	1.01		1/8	.123
	3/16	1.28		5/16	.864	$\frac{3}{4}\times\frac{3}{4}$	3/16	.128
				1/4	.715		1/8	.091

ANGLES WITH UNEQUAL LEGS.—LONG LEG VERTICAL.

Size in inches.	Thickness of metal.	C.	Size in inches.	Thickness of metal.	C.	Size in inches.	Thickness of metal.	C.
$8\frac{1}{4}\times6\frac{1}{4}$	1	82.29	6×4	13/16	35.73	$5\frac{5}{8}\times5$	11/16	28.21
$8\frac{1}{8}\times6\frac{1}{8}$	3/4	62.56		3/4	33.33		7/16	18.72
8×6	1/2	42.83		11/16	30.62	$5\frac{3}{4}\times3\frac{3}{4}$	5/8	24.85
$7\times3\frac{1}{2}$	1	56.42		5/8	28.32	$5\frac{1}{2}\times3\frac{1}{2}$	3/8	14.72
	15/16	53.33		9/16	25.76	5×4	7/8	26.61
	7/8	50.24		1/2	23.04		3/4	23.31
	13/16	47.04		7/16	20.43		5/8	19.89
	3/4	43.50		3/8	17.71		1/2	16.26
	11/16	40.53	$6\times3\frac{1}{2}$	1	41.76		3/8	12.48
	5/8	37.17		15/16	39.52	$5\times3\frac{1}{2}$	7/8	26.02
	9/16	33.76		7/8	37.22		13/16	24.42
	1/2	30.29		13/16	34.93		3/4	22.82
	7/16	26.72		3/4	32.53		11/16	21.17
$6\frac{7}{8}\times4\frac{3}{8}$	15/16	51.09		11/16	30.13		5/8	19.46
$6\frac{1}{2}\times4$	3/8	20.64		5/8	27.68		9/16	17.70
6×4	1	42.77		9/16	25.17		1/2	15.94
	15/16	40.48		1/2	22.61		7/16	14.08
	7/8	38.13		7/16	20.00		3/8	12.21
				3/8	17.33			

For Weight and Properties of these angles, see Tables Chapter XX.

COEFFICIENT OF STRENGTH IN TONS FOR ALL SIZES AND THICKNESSES OF STEEL ANGLES (*continued*).

Reduce 20 per cent. for permanent and live loads.

See explanation at beginning of table.

ANGLES WITH UNEQUAL LEGS.—LONG LEG VERTICAL (*cont'd*).

Size in inches.	Thickness of metal.	*C.*
5×3½	5/16	10.34
5×3	13/16	23.73
	3/4	22.18
	11/16	20.58
	5/8	18.93
	9/16	17.22
	1/2	15.92
	7/16	13.76
	3/8	11.94
	5/16	10.08
4½×3	13/16	19.31
	3/4	18.02
	11/16	16.74
	5/8	15.41
	9/16	14.08
	1/2	12.64
	7/16	11.20
	3/8	9.76
	5/16	8.21
4×3½	13/16	15.57
	3/4	14.66
	11/16	13.65
	5/8	12.53
	9/16	11.46
	1/2	10.29
	7/16	9.17
	3/8	8.00
	5/16	6.72
4×3	13/16	15.30
	3/4	14.29
	11/16	13.28
	5/8	12.26
	9/16	11.14
	1/2	10.08
	7/16	8.96
	3/8	7.78
	5/16	6.56

Size in inches.	Thickness of metal.	*C.*
3¾×2⅝	3/4	12.21
	3/8	6.72
3½×3	13/16	11.73
	3/4	10.93
	11/16	10.58
	5/8	9.38
	9/16	8.58
	1/2	7.73
	7/16	6.88
	3/8	6.02
	5/16	5.12
3½×2½	11/16	9.86
	5/8	9.12
	9/16	8.32
	1/2	7.52
	7/16	6.72
	3/8	5.81
	5/16	4.96
	1/4	4.00
3⅝×2⅛	3/8	5.81
3½×2	1/4	3.84
3¼×2	9/16	6.93
	1/2	6.24
	3/8	4.85
	1/4	3.36
3×2½	9/16	6.13
	1/2	5.54
	7/16	4.96
	3/8	4.32
	5/16	3.68
	1/4	2.98
3×2	1/2	5.33
	7/16	4.74
	3/8	4.16
	5/16	3.52

Size in inches.	Thickness of metal.	*C.*
3×2	1/4	2.88
2½×2	1/2	3.73
	7/16	3.30
	3/8	2.93
	5/16	2.50
	1/4	2.02
	3/16	1.54
2½×1¾	1/4	1.97
	3/16	1.54
2½×1½	3/8	2.77
	3/16	1.49
2½×1¼	3/8	2.66
	3/16	1.44
2¼×1½	1/2	3.14
	3/8	2.24
	1/4	1.60
	3/16	1.22
2×1¾	5/16	1.60
	3/16	1.013
2×1½	5/16	1.546
	3/16	0.960
2×1⅜	1/4	1.226
2—1⅜	3/16	0.960
2 3/16x1 7/16	3/8	1.920
2×1¼	3/16	0.960
1⅜×1⅛	5/16	0.8
1⅜×1⅛	1/8	.320
1⅜×1	1/4	.480
1⅜×1	1/8	.320
1⅜×⅞	1/8	.304
1×⅝	1/8	.149

For Weight and Properties of these angles, see Tables Chapter XX.

COEFFICIENT OF STRENGTH IN TONS FOR ALL SIZES AND THICKNESSES OF STEEL ANGLES (*continued*).

Reduce 20 per cent. for permanent and live loads.

See explanation at beginning of table.

ANGLES WITH UNEQUAL LEGS. SHORT LEG VERTICAL.

Size in inches.	Thickness of metal.	C.
8¼×6¼	1	49.06
8⅛×6⅛	¾	37.33
8 ×6	½	25.60
7×3½	1	15.78
	15/16	14.93
	7/8	14.08
	13/16	13.22
	3/4	12.32
	11/16	11.41
	5/8	10.50
	9/16	9.60
	1/2	8.64
	7/16	7.84
6⅞×4⅜	15/16	21.70
6½×4	3/8	8.64
6×4	1	20.21
	15/16	19.14
	7/8	18.08
	13/16	16.96
	3/4	15.84
	11/16	14.72
	5/8	13.54
	9/16	12.32
	1/2	11.09
	7/16	9.86
	3/8	8.53
6×3½	1	15.46
	15/16	14.61
	7/8	13.81
	13/16	12.96
	3/4	12.10
	11/16	11.25
	5/8	10.34
	9/16	9.44
	1/2	8.48
	7/16	7.52
	3/8	6.56

Size in inches.	Thickness of metal.	C.
5⅝×5	11/16	22.77
	7/16	15.14
5¾×3¾	5/8	11.20
5½×3½	3/8	6.50
5×4	7/8	17.65
	3/4	15.46
	5/8	13.22
	1/2	10.88
	3/8	8.37
5×3½	7/8	13.44
	13/16	12.64
	3/4	11.84
	11/16	10.98
	5/8	10.13
	9/16	9.22
	1/2	8.32
	7/16	7.41
	3/8	6.45
	5/16	5.44
5×3	13/16	9.28
	3/4	8.69
	11/16	8.05
	5/8	7.41
	9/16	6.77
	1/2	6.13
	7/16	5.44
	3/8	4.74
	5/16	4.00
4½×3	13/16	9.12
	3/4	8.53
	11/16	7 94
	5/8	7.30
	9/16	6.66
	1/2	6.02
	7/16	5.38
	3/8	4.69

Size in inches.	Thickness of metal.	C.
4½×3	5/16	4.05
	13/16	12.26
	3/4	11.46
	11/16	10.66
	5/8	9.81
4×3½	9/16	8.96
	1/2	8.10
	7/16	7.20
	3/8	6.29
	5/16	5.38
4×3	13/16	8.96
	3/4	8.37
	11/16	7.78
	5/8	7.20
	9/16	6.56
	1/2	5.97
	7/16	5.28
	3/8	4.64
	5/16	3.94
3¾×2⅝	3/4	6.29
	3/8	3.52
3½×3	13/16	8.80
	3/4	8.21
	11/16	7.68
	5/8	7.09
	9/16	6.45
	1/2	5.86
	7/16	5.22
	3/8	4.53
	5/16	3.84
	11/16	5.28
	5/8	4.90
3½×2½	9/16	4.48
3½×2½	1/2	4.05
	7/16	3.62
	3/8	3.14

For Weight and Properties of these angles, see Tables Chapter XX.

COEFFICIENT OF STRENGTH IN TONS FOR ALL SIZES AND THICKNESSES OF STEEL ANGLES (*continued*).

Reduce 20 per cent. for permanent and live loads.

See explanation at beginning of table.

ANGLES WITH UNEQUAL LEGS.—SHORT LEG VERTICAL (*continued*).

Size in inches.	Thickness of metal.	C.
$3\frac{1}{2}\times2\frac{1}{2}$	$\frac{5}{16}$	2.66
	$\frac{1}{4}$	2.18
$3\frac{5}{8}\times2\frac{1}{8}$	$\frac{3}{8}$	2.18
$3\frac{1}{2}\times2$	$\frac{1}{4}$	1.12
$3\frac{1}{4}\times2$	$\frac{9}{16}$	2.82
	$\frac{1}{2}$	2.56
	$\frac{3}{8}$	1.97
	$\frac{1}{4}$	1.38
$3\times2\frac{1}{2}$	$\frac{9}{16}$	4.37
	$\frac{1}{2}$	3.94
	$\frac{7}{16}$	3.52
	$\frac{3}{8}$	3.09
	$\frac{5}{16}$	2.61
	$\frac{1}{1}$	2.13
3×2	$\frac{1}{2}$	2.50
	$\frac{7}{16}$	2.24
	$\frac{3}{8}$	1.97
	$\frac{5}{16}$	1.70
	$\frac{1}{4}$	1.33

Size in inches.	Thickness of metal.	C.
$2\frac{1}{2}\times2$	$\frac{1}{2}$	2.45
	$\frac{7}{16}$	2.18
	$\frac{3}{8}$	1.92
	$\frac{5}{16}$	1.65
	$\frac{1}{4}$	1.33
	$\frac{3}{16}$	1.06
$2\frac{1}{2}\times1\frac{3}{4}$	$\frac{1}{4}$	1.06
	$\frac{3}{16}$	.80
$2\frac{1}{2}\times1\frac{1}{2}$	$\frac{3}{8}$	1.06
	$\frac{3}{16}$	.586
$2\frac{1}{2}\times1\frac{1}{4}$	$\frac{3}{8}$	.746
	$\frac{3}{16}$	.426
$2\frac{1}{4}\times1\frac{1}{2}$	$\frac{1}{2}$	1.386
	$\frac{3}{8}$	1.066
	$\frac{1}{4}$	.746
	$\frac{3}{16}$	.586
$2\times1\frac{3}{4}$	$\frac{5}{16}$	1.226
	$\frac{3}{16}$	.800

Size in inches.	Thickness of metal.	C.
$2\times1\frac{1}{2}$	$\frac{5}{16}$	0.906
	$\frac{3}{16}$	.586
$2\times1\frac{3}{8}$	$\frac{1}{4}$	.640
	$\frac{3}{16}$	.480
$2\frac{3}{16}\text{x}1\frac{7}{16}$	$\frac{3}{8}$	.906
$2\times1\frac{1}{4}$	$\frac{3}{16}$	.373
$1\frac{3}{8}\times1\frac{1}{8}$	$\frac{5}{16}$	.533
	$\frac{1}{8}$	.213
$1\frac{3}{8}\times1$	$\frac{1}{4}$	.266
	$\frac{1}{8}$	.160
$1\times\frac{7}{8}$	$\frac{1}{8}$	.128
$1\times\frac{5}{8}$	$\frac{1}{8}$	.064

For Weight and Properties of these angles, see Tables Chapter XIII.

SAFE DISTRIBUTED LOADS IN TONS FOR CARNEGIE TEES.

Computed for fibre stress of 16,000 lbs. For permanent and live loads, reduce 20 per cent. See further explanation page 513.

Size. Flange by stem.	Wt. per foot.	*C.*	Span in feet.								
			2	3	4	5	6	7	8	9	10
5 ×3	13.6	6.29	3.15	2.10	1.57	1.26	1.05	0.90	0.79	0.70	0.63
5 ×2½	11.0	4.59	2.29	1.53	1.15	0.92	0.76	0.66	0.57	0.51	0.46
4½×3½	15.8	11.36	5.68	3.79	2.84	2.27	1.89	1.62	1.42	1.26	1.14
4½×3	8.5	4.32	2.16	1.44	1.08	0.86	0.72	0.62	0.54	0.48	0.43
4½×3	10.0	5.01	2.51	1.67	1.25	1.00	0.84	0.72	0.63	0.56	0.50
4½×2½	8.0	2.99	1.49	0.96	0.75	0.60	0.48	0.43	0.37	0.32	0.30
4½×2½	9.3	3.47	1.73	1.16	0.87	0.69	0.58	0.50	0.43	0.39	0.35
4 ×5	15.6	16.53	8.27	5.51	4.13	3.31	2.76	2.36	2.07	1.84	1.65
4 ×5	12.0	12.96	6.48	4.32	3.24	2.59	2.16	1.85	1.62	1.44	1.30
4 ×4½	14.6	13.60	6.80	4.53	3.40	2.72	2.27	1.94	1.70	1.51	1.36
4 ×4½	11.4	10.56	5.28	3.52	2.64	2.11	1.76	1.51	1.32	1.17	1.06
4 ×4	13.7	10.77	5.39	3.59	2.69	2.15	1.80	1.54	1.35	1.20	1.08
4 ×4	10.9	8.75	4.37	2.92	2.19	1.75	1.46	1.25	1.09	0.97	0.87
4 ×3	9.3	4.69	2.35	1.56	1.17	0.94	0.78	0.67	0.59	0.52	0.47
4 ×2½	8.6	3.31	1.65	1.10	0.83	0.66	0.55	0.47	0.41	0.37	0.33
4 ×2½	7.3	2.93	1.47	0.98	0.73	0.59	0.49	0.42	0.37	0.33	0.29
4 ×2	7.9	2.13	1.07	0.71	0.53	0.43	0.36	0.30	0.27	0.24	0.21
4 ×2	6.6	1.81	0.91	0.60	0.45	0.36	0.30	0.26	0.23	0.20	0.18
3½×4	12.8	10.56	5.28	3.52	2.64	2.11	1.76	1.51	1.32	1.17	1.06
3½×4	9.9	8.27	4.13	2.76	2.07	1.65	1.38	1.18	1 03	0.92	0.83
3½×3½	11.7	8.11	4.05	2.70	2.03	1.62	1.35	1.16	1.01	0.90	0.81
3½×3½	9.2	6.35	3.17	2.12	1.59	1.27	1.06	0.91	0.79	0.71	0.63
3½×3	10.9	6.03	3.01	2.01	1.51	1.21	1.00	0.86	0.75	0.67	0.60
3½×3	8.5	4.69	2.35	1.56	1.17	0.94	0.78	0.67	0.59	0.52	0.47
3½×3	7.8	3.84	1.92	1.28	0.96	0.77	0.64	0.55	0.48	0.43	0.38
3 ×4	11.8	10.35	5.17	3.45	2.59	2.07	1.72	1.48	1 29	1.15	1.03
3 ×4	10.6	9.49	4.75	3.16	2.37	1.90	1.58	1.36	1.19	1.05	0.95
3 ×4	9.3	8.37	4.19	2.79	2.09	1.67	1.40	1.20	1.05	0.93	0.84
3 ×3½	10.9	7.95	3.97	2.65	1.99	1.59	1.32	1.14	0.99	0.88	0.79
3 ×3½	9.8	7.31	3.65	2.44	1.83	1.46	1.22	1.04	0.91	0 81	0.73
3 ×3½	8.5	6.45	3.23	2.15	1.61	1.29	1.08	0.92	0.81	0.72	0.65
3 ×3	10.0	5.87	2.93	1.96	1.47	1.17	0.98	0.84	0.73	0.65	0.59

SAFE DISTRIBUTED LOADS IN TONS FOR CARNEGIE TEES (*continued*).

Computed for fibre stress of 16,000 lbs. For permanent and live loads, reduce 20 per cent. For further explanation see page 513.

Size. Flange by stem.	Wt. per foot.	*C.*	Span in feet.								
			2	3	4	5	6	7	8	9	10
3 × 3	9.1	5.39	2.69	1.80	1.35	1.08	0.90	0.77	0.67	0.60	0.54
3 × 3	7.8	4.59	2.29	1.53	1.15	0.92	0.76	0.66	0.57	0.51	0.46
3 × 3	6.6	3.95	1.97	1.32	0.99	0.79	0.66	0.56	0.49	0.44	0.39
3 × 2½	7.2	3.20	1.60	1.07	0.80	0.64	0.53	0.46	0.40	0.36	0.32
3 × 2½	6.1	2.77	1.39	0.92	0.69	0.55	0.46	0.40	0.35	0.31	0.28
2¾ × 2	7.4	4.00	2.00	1.33	1.00	0.80	0.67	0.57	0.50	0.44	0.40
2½ × 3	7.2	4.64	2.32	1.55	1.16	0.93	0.77	0.66	0.58	0.52	0.46
2½ × 3	6.1	4.05	2.03	1.35	1.01	0.81	0.68	0.58	0.51	0.45	0.41
2½ × 2¾	6.7	3.89	1.95	1.30	0.97	0.78	0.65	0.56	0.49	0.43	0.39
2½ × 2¾	5.8	3.20	1.60	1.07	0.80	0.64	0.53	0.46	0.40	0.36	0.32
2½ × 2½	6.4	3.15	1.57	1.05	0.79	0.63	0.52	0.45	0.39	0.35	0.31
2½ × 2½	5.5	2.67	1.33	0.89	0.67	0.53	0.44	0.38	0.33	0.30	0.27
2½ × 1¼	2.9	0.48	0.24	0.16	0.12	0.10					
2¼ × 2¼	4.9	2.24	1.12	0.75	0.56	0.45	0.37	0.32	0.28	0.25	0.22
2¼ × 2¼	4.1	1.71	0.85	0.57	0.43	0.34	0.28	0.24	0.21	0.19	0.17
2 × 2	4.3	1.76	0.88	0.59	0.44	0.35	0.29	0.25	0.22	0.20	0.18
2 × 2	3.7	1.33	0.67	0.44	0.33	0.27	0.22	0.19	0.17	0.15	0.13
2 × 1½	3.1	0.80	0.40	0.27	0.20	0.16	0.13	0.11	0.10		
1¾ × 1¾	3.1	1.01	0.51	0.34	0.25	0.20	0.17	0.14	0.13		
1¾ × 1¼	3.6	0.80	0.40	0.27	0.20	0.16	0.13	0.11	0.10		
1½ × 1½	2.4	0.75	0.37	0.25	0.19	0.15	0.12				
1½ × 1½	1.84	0.59	0.29	0.20	0.15	0.12					
1¼ × 1¼	2.04	0.53	0.27	0.18	0.13	0.11					
1¼ × 1¼	1.53	0.37	0.19	0.12	0.09	0.07					
1 × 1	1.23	0.27	0.13	0.09	0.07	0.05					
1 × 1	0.87	0.16	0.08	0.05	0.04	0.03					

SAFE DISTRIBUTED LOADS IN TONS FOR CAMBRIA AND CARNEGIE Z-BARS.

For fibre stress of 16,000 lbs. per square inch.

To use this table for other spans, or other methods of loading, see explanation page 513.

Size, ins.	Thickness of metal	*C.*	Span in feet.								
			4	5	6	7	8	9	10	12	14
6	3/8	45.0	11.25	9.00	7.50	6.43	5.63	5.00	4.50	3.75	3.21
6 1/16	7/16	52.4	13.11	10.48	8.73	7.48	6.55	5.82	5.24	4.37	3.74
6 1/8	1/2	59.9	14.96	11.97	9.97	8.55	7.48	6.65	5.99	4.99	4.28
6	9/16	61.6	15.40	12.32	10.27	8.80	7.70	6.84	6.16	5.13	4.40
6 1/16	5/8	68.4	17.09	13.67	11.40	9.76	8.55	7.60	6.84	5.70	4.88
6 1/8	11/16	75.2	18.80	15.04	12.53	10.74	9.40	8.36	7.52	6.27	5.37
6	3/4	74.9	18.72	14.98	12.48	10.70	9.36	8.32	7.49	6.24	5.35
6 1/16	13/16	81.2	20.29	16.23	13.53	11.59	10.15	9.02	8.12	6.76	5.80
6 1/8	7/8	87.5	21.86	17.49	14.57	12.49	10.93	9.72	8.75	7.29	6.25
5	5/16	28.5	7.12	5.70	4.75	4.07	3.56	3.17	2.85	2.37	2.03
5 1/16	3/8	34.1	8.52	6.82	5.68	4.87	4.26	3.79	3.41	2.84	2.43
5 1/8	7/16	39.7	9.92	7.94	6.62	5.67	4.96	4.41	3.97	3.31	2.83
5	1/2	40.9	10.24	8.19	6.83	5.85	5.12	4.55	4.09	3.41	2.92
5 1/16	9/16	46.0	11.49	9.19	7.66	6.56	5.75	5.11	4.60	3.83	3.28
5 1/8	5/8	51.1	12.76	10.21	8.51	7.29	6.38	5.67	5.11	4.25	3.65
5	11/16	50.5	12.63	10.10	8.42	7.21	6.32	5.61	5.05	4.21	3.61
5 1/16	3/4	55.2	13.79	11.03	9.19	7.88	6.89	6.13	5.52	4.60	3.94
5 1/8	13/16	59.7	14.94	11.95	9.96	8.54	7.47	6.64	5.97	4.98	4.27
4	1/4	16.8	4.19	3.35	2.79	2.39	2.09	1.86	1.68	1.40	1.20
4 1/16	5/16	20.8	5.21	4.17	3.48	2.98	2.60	2.32	2.08	1.74	1.49
4 1/8	3/8	24.9	6.22	4.98	4.15	3.56	3.11	2.77	2.49	2.08	1.78
4	7/16	25.7	6.44	5.15	4.29	3.68	3.22	2.86	2.57	2.15	1.84
4 1/16	1/2	29.3	7.33	5.87	4.89	4.19	3.67	3.26	2.93	2.44	2.09
4 1/8	9/16	32.9	8.24	6.59	5.49	4.71	4.12	3.66	3.29	2.75	2.35
4	5/8	32.3	8.06	6.45	5.37	4.61	4.03	3.58	3.23	2.69	2.31
4 1/16	11/16	35.5	8.86	7.09	5.91	5.06	4.43	3.95	3.55	2.96	2.53
4 1/8	3/4	38.7	9.68	7.74	6.45	5.53	4.84	4.30	3.87	3.23	2.76
3	1/4	10.3	2.56	2.05	1.71	1.46	1.28	1.14			
3 1/16	5/16	12.7	3.17	2.54	2.12	1.81	1.58	1.41			
3	3/8	13.7	3.44	2.74	2.28	1.96	1.72	1.52			
3 1/16	7/16	15.9	3.97	3.18	2.65	2.27	1.98	1.77			
3	1/2	16.3	4.08	3.26	2.72	2.33	2.04	1.81			
3 1/16	9/16	18.3	4.57	3.66	3.05	2.62	2.28	2.03			

Steel-beam Girders.

A box girder consisting of a pair of steel I-beams with top and bottom flange-plates, furnishes an economical girder for short

spans. The flange-plates are riveted to the beams with rivets $\frac{3}{4}''$ diameter, spaced from 6 to 9 inches on centres. In short girders care must be taken to have a sufficient number of rivets in each plate, between the end of the girder and the centre of the span, to develop the full tensile or compressive strength of the plate, figured at 13,000 lbs. per square inch.

The following tables give the safe loads for the sizes of beams most likely to be used in this way. The values given in the tables are founded upon the moments of inertia of the various sections, deductions being made for the rivet-holes in both flanges. In order to amply compensate for the deterioration of the metal around the rivet-holes from punching, and also because these girders are more often used to support permanent loads, such as brick or stone walls, the maximum fibre stress was limited to 13,000 lbs., although it is but right to state that most of the latest handbooks of the steel manufacturers give tables for such girders based on 15,000 lbs. fibre stress. The author advises that for loads of masonry, which usually come very closely to the estimated load, and which are constantly exerted, the girders be not loaded beyond the values given in the following tables, while for ordinary floor loads, which seldom reach the estimated load, an addition of $\frac{1}{6}$th may be added to the values given in the tables.

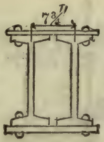
EXAMPLE.—A 13″ brick wall, 15 feet high, is to be built over an opening of 24 feet. What will be the section of the girder required?

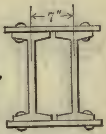
Ans. Assuming 25 feet as the distance, centre to centre of bearings, the weight of the wall will be $25 \times 15 \times 121 = 45{,}375$ lbs., or 22.68 tons.

From the tables we find that a girder composed of two 12″ steel beams, each weighing 31.5 lbs. per foot, and two $14'' \times \frac{1}{2}''$ flange-plates will carry safely, for a span of 25 feet, a uniformly distributed load of 23.23 tons, including its own weight. Deducting the latter, 1.42 tons, given in the next column, we find 21.81 tons for the value of the safe net load, which is 1.07 tons less than required. From the following column we find that by increasing the thickness of the flange-plates $\frac{1}{16}''$ we may add 1.52 tons to the allowable load. This will more than cover the difference. Hence the required section will be two 12″ steel beams 31.5 lbs. per foot, and two $14'' \times \frac{9}{16}''$ steel cover-plates.

STEEL-BEAM BOX-GIRDERS. SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

2–20″ steel I-beams and 2 steel plates 16″ × ¾″.



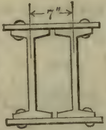
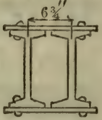


Distance, centre to centre of bearings, in feet.	2 steel plates, 16″ × ¾″; 20″ steel I-beams, 80.0 lbs. per foot.			2 steel plates, 16″ × ¾″; 20″ steel I-beams, 65.0 lbs. per foot.			Increase in weight of girder for 1/16″ increase in thickness of flange-plates.
	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet-heads), in tons of 2,000 lbs.	Increase in safe load for 1/16″ increase in thickness of flange plates.	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet-heads), in tons of 2,000 lbs.	Increase in safe load for 1/16″ increase in thickness of flange-plates.	
10	199.67	1.22	7.22	176.72	1.06	7.34	0.03
11	181.51	1.34	6.56	160.66	1.16	6.68	0.04
12	166.39	1.46	6.02	147.26	1.27	6.12	0.04
13	153.60	1.58	5.56	135.95	1.37	5.65	0.04
14	142.64	1.70	5.16	126.24	1.48	5.25	0.05
15	133.12	1.83	4.81	117.82	1.58	4.90	0.05
16	124.80	1.95	4.51	110.45	1.69	4.59	0.05
17	117.47	2.07	4.25	103.96	1.79	4.32	0.06
18	110.94	2.19	4.01	98.18	1.90	4.08	0.06
19	105.10	2.31	3.80	93.01	2.01	3.86	0.06
20	99.83	2.43	3.61	88.36	2.11	3.67	0.07
21	95.08	2.56	3.44	84.15	2.22	3.50	0.07
22	90.77	2.68	3.28	80.33	2.32	3.34	0.07
23	86.82	2.80	3.14	76.84	2.43	3.19	0.08
24	83.20	2.92	3.01	73.64	2.53	3.06	0.08
25	79.87	3.04	2.89	70.69	2.64	2.94	0.08
26	76.80	3.16	2.78	67.97	2.75	2.82	0.09
27	73.96	3.29	2.68	65.46	2.85	2.72	0.09
28	71.32	3.41	2.58	63.12	2.96	2.62	0.09
29	68.86	3.53	2.49	60.94	3.06	2.53	0.10
30	66.56	3.65	2.41	58.91	3.17	2.45	0.10
31	64.41	3.77	2.33	57.01	3.27	2.37	0.10
32	62.41	3.89	2.26	55.22	3.38	2.29	0.11
33	60.51	4.02	2.19	53.56	3.48	2.22	0.11
34	58.73	4.14	2.12	51.98	3.59	2.16	0.11
35	57.05	4.26	2.06	50.50	3.70	2.10	0.12
36	55.46	4.38	2.01	49.09	3.80	2.04	0.12
37	53.96	4.50	1.95	47.77	3.91	1.98	0.12
38	52.54	4.62	1.90	46.51	4.01	1.93	0.13
39	51.20	4.75	1.85	45.32	4.12	1.88	0.13

Above values are based on maximum fibre strain of 13,000 lbs. per square inch, rivet-holes in both flanges deducted. Weights of girders correspond to lengths, centre to centre of bearings.

STEEL-BEAM BOX-GIRDERS. SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

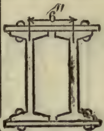
2–18″ steel I-beams and 2 steel plates 16″×¾″.

Distance, centre to centre of bearings, in feet.	2–18″ beams. 70 lbs. per foot.  2–16×¾″ steel plates.			2–18″ beams. 55 lbs. per foot.  2–16×¾″ steel plates.				Add to weight of girder for 1/16″ increase in thickness of plates.
	Safe load in tons, including weight of girder.	Weight of girder in pounds.	Add to safe load for 1/16″ increase in thickness of plates.	Safe load in tons, including weight of girder.	Weight of girder in pounds.	Add to safe load for 5 pounds increase in weight of beam.	Add to safe load for 1/16″ increase in thickness of plates.	
12	132.2	2712	5.43	123.0	2352	2.81	5.43	82
13	122.0	2933	5.01	113.5	2548	2.61	5.01	88
14	113.3	3164	4.66	105.3	2744	2.43	4.66	95
15	105.7	3390	4.35	98.3	2940	2.27	4.35	102
16	99.1	3616	4.07	92.2	3136	2.12	4.07	109
17	93.3	3842	3.83	86.8	3332	2.00	3.83	116
18	88.1	4068	3.62	82.0	3528	1.90	3.62	122
19	83.5	4294	3.43	77.6	3724	1.80	3.43	129
20	79.3	4520	3.26	73.8	3920	1.70	3.26	136
21	75.5	4746	3 10	70.2	4116	1.62	3.10	143
22	72.1	4972	2.96	67.0	4312	1.54	2.96	150
23	69.0	5198	2.83	64.1	4508	1.47	2.83	156
24	66.1	5424	2.72	61.5	4704	1.41	2.72	163
25	63.5	5650	2.61	59.0	4900	1.36	2.61	170
26	61.0	5876	2.51	56.7	5096	1.30	2.51	177
27	58.8	6102	2.41	54.6	5292	1.26	2.41	184
28	56.6	6328	2.33	52.7	5488	1.21	2.33	190
29	54.7	6554	2.25	50.9	5684	1.17	2.25	197
30	52.9	6780	2.17	49.2	5880	1.13	2.17	204
31	51.8	7006	2.10	47.6	6076	1.10	2.10	211
32	49.6	7232	2.04	46.1	6272	1.06	2.04	218
33	48.1	7458	1.98	44.7	6468	1.03	1.98	224
34	46.7	7684	1.92	43.4	6664	1.00	1.92	231
35	45.3	7910	1.86	42.1	6860	.97	1.86	238
36	44.1	8136	1.81	41.0	7056	.94	1.81	245
37	42.9	8362	1.76	39.9	7252	.92	1.76	252
38	41.2	8588	1.72	38.8	7448	.90	1.72	258

Above values are based on maximum fibre strain of 13,000 lbs. per sq. in., rivet-holes in both flanges deducted. Weights correspond to lengths, centre to centre of bearings.

STEEL-BEAM BOX-GIRDERS. SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

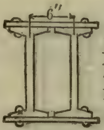
2–15″ steel I-beams and 2 steel plates 14″×⅝″.

Distance, centre to centre of bearings, in feet.	 6¾″ 15″ steel I. 75.0 lbs. per foot. Steel plates, 14″×⅝″.		 6½″ 15″ steel I. 60.0 lbs per foot. Steel plates, 14″×⅝″.		 6″ 15″ steel I. 42.0 lbs. per foot. Steel plates, 14″×⅝″.		Increase in safe load for 1/16″ increase in thickness of flange-plates.	Increase in weight of girder for 1/16″ increase in thickness of flange-plates.
	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet-heads), in tons of 2,000 lbs.	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet-heads), in tons of 2,000 lbs.	Safe load, uniformly distributed (including weight of girder), in ton of 2,000 lbs.	Weight of girder (including rivet-heads), in tons of 2,000 lbs.		
10	122.33	1.06	111.01	0.91	90.29	0.72	4.63	0.03
11	111.21	1.17	100.92	1.00	82.08	0.79	4.21	0.03
12	101.95	1.27	92.51	1.09	75.24	0.86	3.86	0.03
13	94.10	1.38	85.40	1.18	69.45	0.93	3.57	0.04
14	87.38	1.48	79.30	1.27	64.50	1.00	3.31	0.04
15	81.56	1.59	74.01	1.36	60.19	1.08	3.09	0.04
16	76.46	1.70	69.38	1.45	56.43	1.15	2.90	0.05
17	71.96	1.80	65.30	1.54	53.11	1.22	2.72	0.05
18	67.96	1.91	61.67	1.63	50.16	1.29	2.57	0.05
19	64.39	2.01	58.43	1.72	47.52	1.36	2.43	0.05
20	61.17	2.12	55.50	1.81	45.14	1.44	2.32	0.06
21	58.25	2.22	52.86	1.90	42.99	1.51	2.21	0.06
22	55.60	2.33	50.46	2.00	41.04	1.58	2.11	0.06
23	53.19	2.43	48.27	2.09	39.25	1.65	2.02	0.07
24	50.97	2.54	46.25	2.18	37.62	1.72	1.93	0.07
25	48.93	2.65	44.40	2.27	36.12	1.79	1.85	0.07
26	47.05	2.76	42.70	2.36	34.72	1.87	1.78	0.08
27	45.31	2.86	41.12	2.45	33.44	1.94	1.71	0.08
28	43.69	2.96	39.65	2.54	32.25	2.01	1.66	0.08
29	42.18	3.07	38.28	2.63	31.13	2.08	1.60	0.08
30	40.78	3.17	37.00	2.72	30.09	2.15	1.54	0.09
31	39.46	3.23	35.81	2.81	29.12	2.23	1.49	0.09
32	38.23	3.38	34.69	2.80	28.21	2.30	1.45	0.09
33	37.07	3.46	33.64	2.99	27.36	2.37	1.41	0.10
34	35.98	3.60	32.65	3.08	26.55	2.44	1.37	0.10
35	34.95	3.70	31.72	3.17	25.80	2.51	1.33	0.10
36	33.98	3.81	30.84	3.27	25.08	2.58	1.29	0.10
37	33.06	3.91	30.00	3.36	24.40	2.66	1.25	0.11
38	32.20	4.02	29.21	3.45	23.76	2.73	1.22	0.11
39	31.37	4.13	28.47	3.54	23.15	2.80	1.19	0.11

Above values are based on maximum fibre strains of 13,000 lbs. per sq. in., rivet-holes in both flanges deducted. **Weights of girders correspond to lengths, centre to centre of bearings.**

STEEL BEAM BOX-GIRDERS. SAFE LOAD IN TONS, UNIFORMLY DISTRIBUTED.

2–12″ steel I-beams and 2 steel plates 14″×½″.



2 steel plates, 14″×½″. 12′steel I-beams, 40.0 lbs. per foot.

2 steel plates, 14″×½″. 12″ steel I-beams, 31.5 lbs. per foot.

Distance, centre to centre of bearings, in feet.	2 steel plates, 14″×½″; 12′ steel I-beams, 40.0 lbs. per foot. — Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet-heads), in tons of 2,000 lbs.	Increase in safe load for 1/16″ increase in thickness of flange-plates.	2 steel plates, 14″×½″; 12″ steel I-beams, 31.5 lbs. per foot. — Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet-heads), in tons of 2,000 lbs.	Increase in safe load for 1/16″ increase in thickness of flange-plates.	Increase in weight of girder for 1/16″ increase in thickness of flange-plates.
10	64.94	0.65	3.75	58.08	0.57	3.81	0.03
11	59.02	0.71	3.40	52.80	0.63	3.45	0.03
12	54.12	0.78	3.12	48.40	0.68	3.17	0.03
13	49.95	0.84	2.88	44.68	0.74	2.93	0.04
14	46.39	0.91	2.68	41.48	0.80	2.72	0.04
15	43.29	0.97	2.50	38.72	0.85	2.53	0.04
16	40.59	1.04	2.34	36.30	0.91	2.38	0.05
17	38.20	1.10	2.21	34.16	0.97	2.24	0.05
18	36.08	1.17	2.03	32.27	1.03	2.11	0.05
19	34.18	1.23	1.97	30.57	1.08	2.00	0.05
20	32.47	1.30	1.87	29.04	1.14	1.90	0.06
21	30.93	1.36	1.78	27.66	1.20	1.81	0.06
22	29.52	1.43	1.70	26.40	1.25	1.73	0.06
23	28.23	1.49	1.63	25.25	1.31	1.65	0.07
24	27.06	1.56	1.56	24.20	1.37	1.58	0.07
25	25.98	1.62	1.50	23.23	1.42	1.52	0.07
26	24.98	1.69	1.44	22.34	1.48	1.46	0.08
27	24.05	1.75	1.38	21.51	1.54	1.41	0.08
28	23.19	1.82	1.34	20.74	1.60	1.36	0.08
29	22.39	1.88	1.29	20.03	1.65	1.31	0.08
30	21.65	1.95	1.25	19.36	1.71	1.27	0.09
31	20.95	2.01	1.21	18.73	1.77	1.23	0.09
32	20.29	2.08	1.17	18.15	1.82	1.19	0.09
33	19.68	2.14	1.14	17.60	1.88	1.15	0.10
34	19.10	2.21	1.10	17.08	1.94	1.12	0.10
35	18.55	2.27	1.07	16.59	1.99	1.09	0.10
36	18.04	2.34	1.04	16.13	2.05	1.06	0.10
37	17.55	2.40	1.01	15.70	2.11	1.03	0.11
38	17.09	2.47	0.99	15.28	2.17	1.00	0.11
39	16.65	2.53	0.96	14.89	2.22	0.98	0.11

Above values are based on maximum fibre strains of 13,000 lbs. per sq. in., rivetholes in both flanges deducted. Weights of girders correspond to lengths, centre to centre of bearings.

STEEL-BEAM BOX-GIRDERS. SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

2–10″ steel I-beams and 2 steel plates 12″ × ½″.

Distance, centre to centre of bearings, in feet.	2 steel plates, 12″ × ½″  10″ steel I-beams, 35.0 lbs. per foot.			2 steel plates, 12″ × ½″  10″ steel I-beams, 25.0 lbs. per foot.			Increase in weight of girder for 1/16″ increase in thickness of flange-plates.
	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet heads), in tons of 2,000 lbs.	Increase in safe load for 1/16″ increase in thickness of flange-plates.	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet heads), in tons of 2,000 lbs.	Increase in safe load for 1/16″ increase in thickness of flange-plates.	
10	44.35	0.55	2.59	39.23	0.47	2.64	0.02
11	40.32	0.60	2.36	35.66	0.52	2.40	0.03
12	36.96	0.65	2.16	32.69	0.56	2.20	0.03
13	34.12	0.71	1.99	30.18	0.61	2.03	0.03
14	31.68	0.76	1.85	28.02	0.66	1.89	0.03
15	29.57	0.82	1.73	26.15	0.71	1.76	0.04
16	27.72	0.87	1.62	24.52	0.75	1.65	0.04
17	26.09	0.93	1.52	23.08	0.80	1.55	0.04
18	24.64	0.98	1.44	21.79	0.85	1.47	0.04
19	23.34	1.04	1.36	20.65	0.89	1.39	0.05
20	22.18	1.09	1.30	19.62	0.94	1.32	0.05
21	21.12	1.15	1.23	18.68	0.99	1.26	0.05
22	20.16	1.20	1.18	17.83	1.04	1.20	0.05
23	19.28	1.26	1.13	17.06	1.08	1.15	0.06
24	18.48	1.31	1.08	16.35	1.13	1.10	0.06
25	17.74	1.36	1.04	15.69	1.18	1.06	0.06
26	17.06	1.42	1.00	15.09	1.22	1.02	0.06
27	16.43	1.47	0.96	14.53	1.27	0.98	0.07
28	15.84	1.53	0.93	14.01	1.32	0.94	0.07
29	15.29	1.58	0.89	13.53	1.37	0.91	0.07
30	14.78	1.64	0.86	13.08	1.41	0.88	0.07
31	14.31	1.69	0.84	12.65	1.46	0.85	0.08
32	13.86	1.75	0.81	12.26	1.51	0.82	0.08
33	13.44	1.80	0.78	11.89	1.55	0.80	0.08
34	13.04	1.86	0.76	11.54	1.60	0.78	0.08
35	12.67	1.91	0.74	11.21	1.65	0.75	0.09
36	12.32	1.96	0.72	10.90	1.70	0.73	0.09
37	11.99	2.02	0.70	10.60	1.74	0.71	0.09
38	11.67	2.07	0.68	10.32	1.79	0.69	0.09
39	11.37	2.13	0.66	10.06	1.84	0.67	0.10

Above values are based on maximum fibre strains of 13,000 lbs. per square inch, rivet-holes in both flanges deducted. Weights of girders correspond to lengths, centre to centre of bearings

BEAMS SUPPORTING BRICK WALLS.

In calculating the size of a girder to support a brick wall, the structure of the wall should be carefully considered. If the wall is without openings, and does not support floor beams, only the

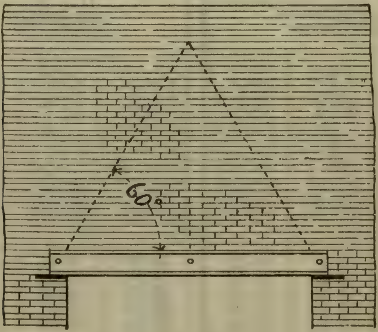


Fig. 10

portion of the wall included within the dotted lines, Fig. 10, need be considered as being supported by the girder. The beams in that case, however, should be made very stiff, so as to have little deflection. If there are several openings above the girder, and especially if there be a pier over the centre of the girder, as shown in Fig. 11, then the manner in which the weight bears on the girder should be carefully considered. In a case such as this the entire dead weight included between the dotted lines A A and B B should be considered as coming on the girder, and proper allowance made for the load being mostly concentrated at the centre.

When beams are used to support a wall entirely (that is, the beams run under the whole length of the wall), and the wall is more than sixteen or eighteen feet long, the whole weight of the wall should be taken as coming upon the beams; for, if the beams should bend, the wall would settle, and might push out the supports, and thus cause the whole structure to fall.

FRAMING AND CONNECTING STEEL BEAMS.

Separators.—When beams are used to support walls, or as girders to carry floor beams, they are often placed side by side; and should in such cases be connected by means of bolts, and cast-iron separators fitting closely between the flanges of the beams. The office of these separators is, in a measure, to hold in position the compression flanges of the beams, preventing side

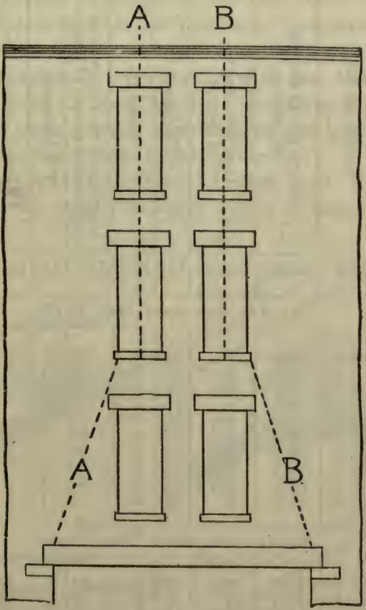
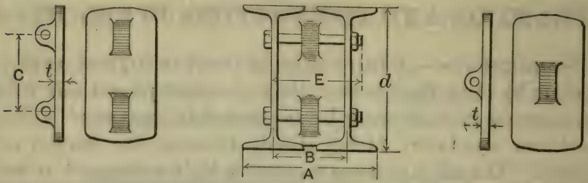


Fig. 11

deflection or buckling, and also to unite the beams so as to cause them to act in unison as regards vertical deflection. Separators should be provided near the supports and at points where heavy loads are imposed, otherwise at regular intervals of from 5 to 6 feet.

The illustrations on the following page show a pair of beams connected by separators, and also the pattern of separators now

CAST-IRON SEPARATORS FOR I-BEAMS.



Section No.	Beams. Depth. d	Weight per foot.	Out to out of flanges of beams. A	Centre to centre of beams. B	Separators. Thickness. t	Weight.	Increase of wt. for each inch additional spread of beams.	Bolts, square heads and hex nuts. Diameter.	Centre to centre of bolts. C	Length. E	Weight of bolts and nuts.	Inc. of wt. of bolts for each in. additional spread of beams.
	Ins.	Lbs.	Inches	Inches	In.	Lbs.	Lbs.	In.	Ins.	Ins.	Lbs.	Lbs.
Separators with One Bolt.												
B 5	3	5.5	5 5/16	3	⅜	1.1	.29	⅝	....	3⅞	.94	.085
B 9	4	7.5	5⅞	3¼	⅜	1.6	.38	¾	....	4¼	.98	.123
B 13	5	9.75	6½	3½	⅜	2.0	.49	¾	....	4½	1.01	.123
B 17	6	12.25	7 5/16	4	½	3.3	.78	¾	....	5	1.07	.123
B 21	7	15.0	7⅞	4¼	½	3.9	.92	¾	....	5¼	1.10	.123
B 25	8	17.75	8½	4½	½	4.7	1.06	¾	....	5⅝	1.15	.123
B 29	9	21.0	9 5/16	5	½	5.9	1.20	¾	....	6⅛	1.21	.123
B 33	10	25.0	9⅞	5¼	½	6.8	1.33	¾	....	6⅜	1.24	.123
B 41	12	31.5	10¾	5¾	½	8.8	1.61	¾	....	6⅞	1.30	.123
B 105	12	40.0	11¼	6	½	8.9	1.58	¾	....	7¼	1.35	.123
Separators with Two Bolts.												
B 41	12	31.5	10¾	5¾	½	9.5	1.61	¾	6½	6⅞	2.61	.246
B 105	12	40.0	11¼	6	½	9.5	1.58	¾	6½	7¼	2.70	.246
B 53	15	42.0	11¾	6¼	½	12.5	2.02	¾	7	7½	2.76	.246
B 109	15	60.0	12¾	6¾	½	13.0	1.97	¾	7	8⅛	2.92	.246
B 113	15	80.0	13⅝	7¼	½	13.2	1.91	¾	7	8⅞	3.10	.246
B 65	18	55.0	12¾	6¾	⅝	19.8	2.41	¾	9	8	2.89	.246
B 73	20	65.0	13¼	7	⅝	22.9	3.37	⅞	10	8⅜	4.20	.334
B 121	20	80.0	14¾	7¾	⅝	24.6	3.34	⅞	10	9¼	4.49	.334
B 89	24	80.0	14¾	7¾	⅝	30.3	4.07	⅞	12	9⅛	4.45	.334

Lengths and weights of separator bolts in above tables are for girders composed of two beams of minimum section as shown. Lengths of bolts for intermediate and maximum sizes of beams may be obtained by adding twice the increase of web thickness to the lengths given.

in common use. The table below the cuts gives the various dimensions and weights as adopted by the Cambria Steel Co. The weights vary slightly from those given by other manufacturers, but the table is probably as near an average of current practice as any that could be given.

Separators formed of pieces of steam or gas pipe, cut to the desired length and slipped over the bolts are often used by contractors. Such separators permit the beams to act independently of each other, and should not be used in any place where one beam is liable to receive a greater load than the other, and as this condition occurs in almost every case where two or more beams are used together, it follows that "cast-iron separators, made to fit the space between the beams," should be specified in almost every instance. Separators with two bolts should be used for beams over 12 ins. in depth. For 12-inch beams either one or two bolts may be used, according to the load; for beams under 12 inches in height one bolt is sufficient.

Beam Connections.—Steel beams and channels are framed together by means of short pieces of angle-bars, which

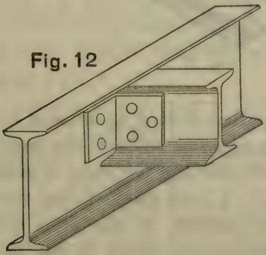


Fig. 12

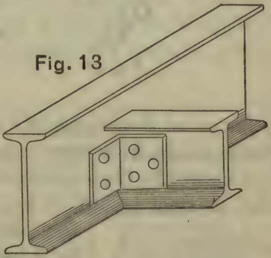


Fig. 13

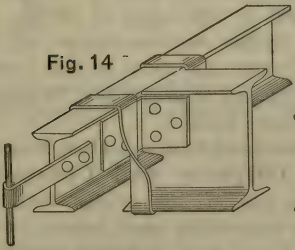


Fig. 14

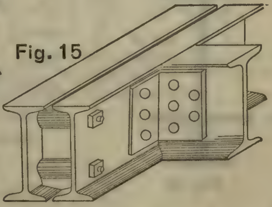


Fig. 15

are usually riveted to the floor or tail beam and bolted to the girder. The angles are always used in pairs, one on each side of the floor beam.

If the floor beam is framed flush, either with the top or bottom of the girder, or where two beams of the same height are framed together, the end of the beam supported should be "coped," or cut to fit the shape of the girder or supporting beam. The maximum clearance space allowed between end of beam and web varies from $\frac{1}{16}$ inch in the smaller beams to $\frac{1}{8}$ inch in the larger ones.

Figs. 12 to 14 show beams of different depths framed in different positions, and Fig 15. shows a beam framed into a girder of the same depth.

When a floor beam rests on top of another beam or girder, as in Fig. 17, the beam should be secured by means of a pair of wrought-iron clips, shown in Fig. 16, shaped so as to fit closely

Fig. 16

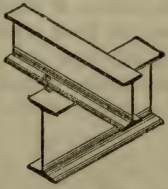


Fig. 17

the top flange of the girder, and either bolted or riveted to the lower flange of the floor beam, on opposite sides of the same.

Fig. 18 shows the best method of framing the ends of wooden floor joist to steel beams, a $4 \times 3 \times \frac{1}{2}$ inch angle being riveted the whole length of the steel beam, by $\frac{3}{4}$-inch rivets, about 6 ins. apart. The top of the beam is usually secured by iron dogs or clamps. If the floor beams are over 3 feet apart, short lengths of angles may be placed under each beam.

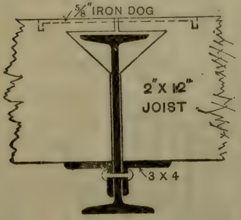


Fig. 18

Standard Connection Angles for I-Beams and Channels.—The size of the angles and the number of the rivets used for connecting steel beams, varies somewhat with different shops and with different structural engineers, so that there cannot be said to be a universal standard. The variations in the different "standards," however, are not very great, and as the connections adopted by the Carnegie

Steel Co. are perhaps the most used, the author has selected them for illustration on the following pages.

These connections are designed on the basis of an allowable shearing stress of 10,000 lbs. per square inch, and a bearing stress of 20,000 lbs. per square inch on rivets or bolts, corresponding with extreme fibre stresses in the I-beams of 16,000 lbs. per square inch. The number of rivets or bolts required is found to be dependent, in most instances, on their bearing values.

The connections have been proportioned with a view to covering most cases occurring in ordinary practice, with the usual relations of depth of beam to length of span. In extreme instances, however, where beams of short relative span lengths are loaded to their full capacity, or when beams frame opposite each other into another beam with web thickness less than 9/16 inch, it may be found necessary to make provision for additional strength in the connections. The limiting span lengths, at and above which the standard connection angles may be used with perfect safety, are given in the following table:

TABLE OF MINIMUM SPANS OF I-BEAMS, FOR WHICH STANDARD CONNECTION ANGLES MAY BE SAFELY USED WITH BEAMS LOADED TO THEIR FULL CAPACITY.

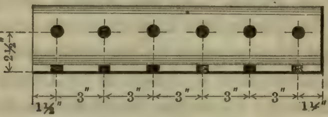
Designation of beam.	Minimum safe span, in feet.	Designation of beam.	Minimum safe span, in feet.	Designation of beam.	Minimum safe span, in feet.
24″—80 lbs.	21.0	15″—42 lbs.	10.5	7″—15 lbs.	
20 80 "	18.0	12 40 "	9.0	6 12.25 lbs.	
20 65 "	17.0	12 31.5 lbs.	7.5	5 9.75 lbs.	
18 55 "	14.0	10 25 lbs.	9.5	4 7.5 lbs.	
15 80 "	16.0	9 21 "	8.0	3 55 "	
15 60 "	12.5	8 18 "	6.5		

For shorter spans and for heavier beams the number of rivets required should be computed by the method explained in Chapter XII.

STANDARD CONNECTION ANGLES FOR I-BEAMS AND CHANNELS.*

S | F
6 | 12

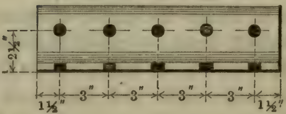
4" x 4" x 3/8" L—1' 6" lg. Weight 38.4 lbs.



for 24" I's

S | F
5 | 10

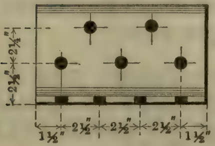
4" x 4" x 3/8" L—1' 3" lg. Weight 32 lbs.



for 18" and 20" I's

S | F
5 | 8

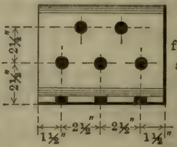
6" x 4" x 3/8" L—0' 10½" lg.
Weight 28 lbs.



for 15" I's and ⊏'s

S | F
5 | 6

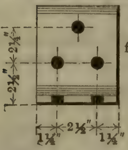
6" x 4" x 3/8" L—0' 8" lg.
Weight 21.9 lbs.



for 12" I's and ⊏'s

S | F
3 | 4

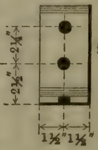
6" x 4" x 3/8" L—0' 5" lg.
Weight 13.8 lbs.



for 7", 8", 9" and 10" I's and ⊏'s

S | F
2 | 2

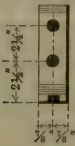
6" x 4" x 3/8" L—0' 3" lg.
Weight 8.2 lbs.



for 5" and 6" I's and ⊏'s

S | F
2 | 2

6" x 4" x 3/8" L—0' 1¾" lg.
Weight 5.6 lbs.



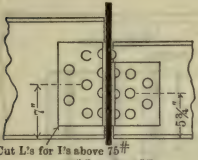
for 3" and 4" I's and ⊏'s

S | F
Shop | Field
Rivets or Bolts.

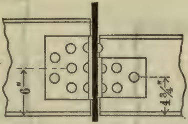
All holes for ¾" Bolts or Rivets.
The weights of connections include shop and field rivets.
2½" gauge on all 4" legs.

* As adopted by the Carnegie Steel Co., Ltd.

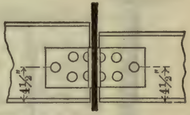
CONNECTIONS FOR DIFFERENT DEPTHS OF BEAMS FRAMING OPPOSITE.*



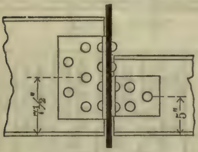
15" I and 12" I



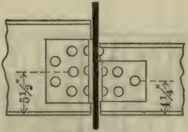
12" I and 10" I



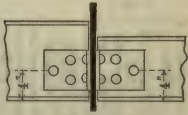
10" I and 9" I



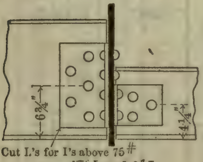
15" I and 10" I



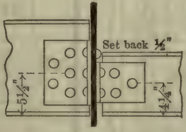
12" I and 9" I



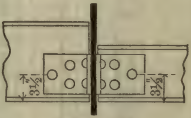
10" I and 8" I
9" I and 8" I



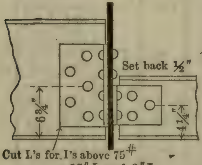
15" I and 9" I



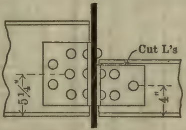
12" I and 8" I



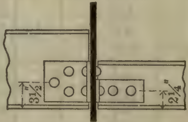
10" I and 7" I
9" I and 7" I
8" I and 7" I



15" I and 8" I

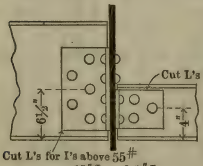


12" I and 7" I

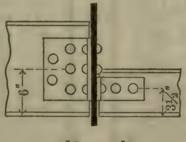


10" I and 6" I
9" I and 6" I
8" I and 6" I
7" I and 6" I

Standard connection angles to be used on larger beams unless gauge exceeds 2¾" on special angles of smaller beams.



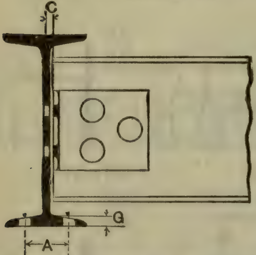
15" I and 7" I

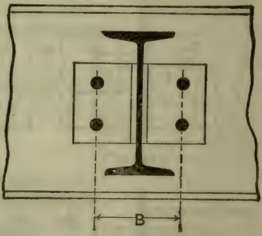


12" I and 6" I

* As adopted by the Carnegie Steel Co., Ltd.

STANDARD SPACING AND DIMENSIONS OF RIVET AND BOLT HOLES THROUGH FLANGES AND CONNECTION ANGLES OF I-BEAMS.



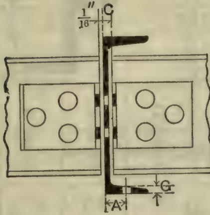


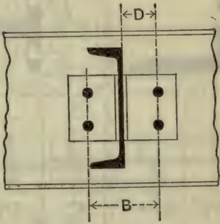
Depth in inches.	Weight per foot.	Max. size bolt or rivet in flanges.	Gauge in inches. A	Std. Conn. spacing. B	Distance in ins.* C	Grip in ins. G
	100			5¾	7/16	
	95			5 11/16	7/16	
24	90	1	4	5⅝	⅜	27/32
	85			5 9/16	⅜	
	80			5½	5/16	
	100			5⅞	½	
	95			5 13/16	½	
	90			5¾	7/16	
20	85	⅞	4	5 11/16	7/16	29/32
	80			5⅝	⅜	
	75			5 11/16	7/16	
	70			5 9/16	⅜	
	65		3½	5½	5/16	25/32
	70			5¾	7/16	
	65			5⅝	⅜	
18	60	⅞	3¼	5 9/16	⅜	11/16
	55			5½	5/16	
	100	⅞		6 3/16	11/16	
	95			6 1/16	⅝	
	90			6	9/16	
	85			5⅞	½	
	80		3¾	5 13/16	½	1 1/32
	75	¾		5⅞	½	
15	70			5¾	7/16	
	65			5 11/16	7/16	
	60		3¼	5⅝	⅜	13/16
	55			5⅝	⅜	
	50			5 9/16	⅜	
	45			5½	5/16	
	42	¾	3	5 7/16	¼	⅝
12	55	¾	3	5 13/16	½	
	50			5 11/16	7/16	

Depth in inches.	Weight per foot.	Max. size bolt or rivet in flanges.	Gauge in inches. A	Std. Conn. spacing. B	Distance in inches. C	Grip in inches. G
	45			5 9/16	⅜	
12	**40**		3	5½	5/16	21/32
	35	¾		5 7/16	5/16	
	31.5		2¾	5⅜	¼	17/32
	40			5¾	7/16	
	35			5⅝	⅜	
10	30	¾	2⅝	5½	5/16	15/32
	25			5 5/16	¼	
	35			5¾	7/16	
9	30	¾	2½	5 9/16	⅜	7/16
	25			5 7/16	5/16	
	21			5 5/16	¼	
	25.5			5 9/16	⅜	
	23			5 7/16	5/16	
8	20.5	¾	2¼	5⅜	¼	13/32
	18			5 5/16	3/16	
	20			5½	5/16	
7	17.5	⅝	2¼	5⅜	¼	⅜
	15			5¼	3/16	
	17.25			5½	5/16	
6	14.75	⅝	2	5⅜	¼	11/32
	12.25			5¼	3/16	
	14.75			5½	5/16	
5	12.25	½	1¾	5⅜	¼	5/16
	9.75			5¼	3/16	
	10.5			5 7/16	5/16	
	9.5	½	1½	5⅜	¼	9/32
4	8.5			5¼	3/16	
	7.5			5 3/16	3/16	
	7.5			5⅜	¼	
3	6.5	⅜	1 7/16	5¼	3/16	¼
	5.5			5 3/16	3/16	

Weights in heavy print are standard, others are special.
All holes for connections to be for ¾″ rivets of bolts.
* From centre of through beam to end of tail beam.

STANDARD SPACING AND DIMENSIONS OF RIVET AND BOLT HOLES THROUGH FLANGES AND CONNECTION ANGLES OF CHANNELS.*





Depth in ins.	Weight per ft.	Max. size bolt or rivet in flanges.	Gauge in ins. A	Std. Conn. spacing. B	Distance in inches. C	Grip in ins. G
15	55			5 13/16	7/8	
	50			5¾	13/16	
	45	¾	2¼	5⅝	11/16	⅝
	40			5½	⅝	
	35			5 7/16	½	
	33		1⅞	5⅜	½	21/32
12	40			5¾	13/16	
	35			5⅝	11/16	
	30	¾	2	5½	9/16	15/32
	25			5⅜	7/16	
	20.5		1¾	5 5/16	⅜	
10	35			5 13/16	⅞	
	30			5 11/16	¾	
	25	¾	2	5½	⅝	⅜
	20			5⅜	7/16	
	15		1½	5¼	5/16	7/16
9	25			5⅝	11/16	
	20			5½	½	⅜
	15	¾	1¾	5 5/16	⅜	
	13.25		1⅜	5¼	5/16	13/32
8	21.25			5 9/16	⅝	
	18.75			5½	9/16	
	16.25	¾	1½	5⅜	7/16	⅜
	13.75			5 5/16	⅜	
	11.25		1¼	5¼	5/16	
7	19.75			5⅛	11/16	
	17.25		1½	5½	9/16	⅜
	14.75	⅝		5 7/16	½	
	12.25			5 5/16	⅜	
	9.75		1¼	5 3/16	¼	11/32
6	15.5			5 9/16	⅝	
	13.0		1⅜	5 7/16	½	
	10.5	⅝		5 5/16	⅜	11/32
	8.0		1⅛	5 3/16	¼	
5	11.5			5½	9/16	
	9.0	½	1¼	5 5/16	⅜	5/16
	6.5		1	5 3/16	¼	
4	7.25			5 5/16	⅜	
	6.25	½	1	5¼	5/16	9/32
	5.25			5 3/16	¼	
3	6.0			5⅜	7/16	
	5.0	⅜	1 5/16	5¼	5/16	¼
	4.0			5 3/16	¼	

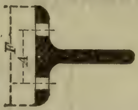
STANDARD AND SPECIAL ANGLES.

Depth of leg in ins.	Max. diam. of bolt or rivet.	Gauge in inches D
8	1	Variable
7	1	"
6	1	"
5	1	"
4½	1	"
4	1	"
3½	1	2
3¼	⅞	1¾
3	⅞	1¾
2¾	¾	1½
2½	¾	1⅜
2¼	¾	1¼
2	⅝	1⅛
1¾	⅝	15/16
1½	½	13/16
1⅜	½	¾
1¼	½	11/16
1	⅜	9/16
⅞	⅜.	½
¾	¼	7/16

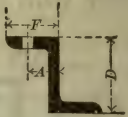
Weights in heavy print are standard, others are special.
All holes for connections to be for ¾" rivets or bolts.
*Adopted by the Carnegie Steel Co., Ltd.

STANDARD GAUGES FOR ANGLES, TEES AND Z-BARS.

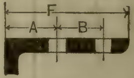
Flange F.	Gauge A.	Rivet or bolt.
7″	3½″	⅞ or 1″
6	3½	" 1
5	2¼-3	" 1
4½	2½	" 1
4	2¼-2½	" 1
3½	2	" 1
3¼	1¾	¾ or ⅞
3	1¾	" ⅞
2¾	1½-1⅝	¾
2½	1⅜	¾
2¼	1¼	¾
2	1⅛	⅝
1¾	1⁵⁄₁₆	⅝
1½	1³⁄₁₆-⅞	½
1⅜	¾	½
1¼	1¹⁄₁₆	½
1⅛	⅝	½
1¹⁄₁₆	⅝	⅜
1	⁹⁄₁₆	⅜
⅞	½	⅜
¾	⁷⁄₁₆	¼
⅝	⅜	³⁄₁₆



Flange F.	Gauge A.	Rivet or bolt.
5″	2⅜″	¾″
4½	2⅛	¾
4	1¹⁵⁄₁₆	¾
3½	1¹¹⁄₁₆	⅝
3	1⁵⁄₁₆	⅝
2¾	1¼	½
2½	1³⁄₁₆	½
2¼	1⅛	½
2	1	½
1¾	⅞	½
1½	¾	⅜
1¼	⅝	¼
1	½	³⁄₁₆

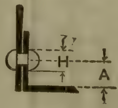


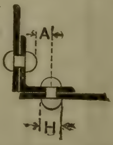
Depth D.	Flange F.	Gauge A.	Rivet or bolt.
6″	3½″	2″	1″
5	3¼	1¾	⅞
4	3	1¾	⅞
3	2¾	1½	¾



F.	A.	B.	Rivet.
7	2½ 2	3 3½	⅞
6½	2½ 2	2½ 3	⅞
6	2½ 2	2¼ 2½	¾ or ⅞
5½	2¼ 2	2 2¼	¾ or ⅞
5 *	2 1¾	1¾ 2	¾ or ⅞
4½	2 1¾	1¼ 1½	¾ or ⅞

MINIMUM GAUGE FOR MACHINE RIVETING.

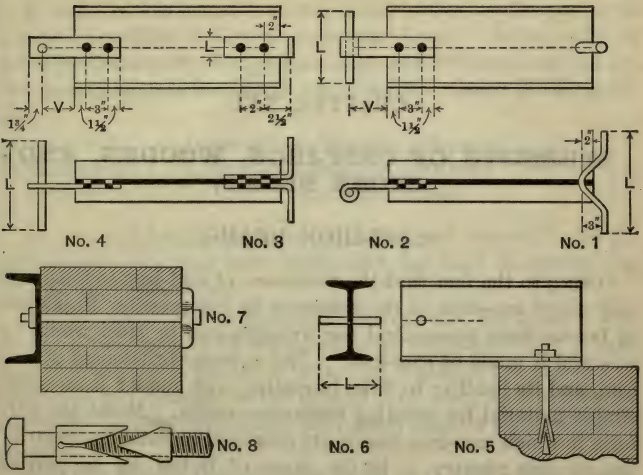




A must not be less than ¼″+½H. For size of rivet-heads, see page 373.

* For 4½″ and 5″ legs rivet-holes should be staggered, so that the distance between centres of rivets shall not be less than 2″ for ¾″ rivets, or 2¼″ for ⅞″ rivets.

WALL ANCHORS FOR BEAMS AND CHANNELS.



No. of anchor.	Depth of beam or channel in inches.	Size of bar or angles in inches.	Length L in inches.	Weight in pounds.
1	3 to 9 10 to 24	¾	15 18	2¼ 2¾
2	3 to 5 6 to 10 12 to 24	2¼×5/16 3 ×⅜ 3 ×⅜	6 6 12	V
3	3 to 5 6 to 10 12 to 24	6×4×⅜ 6×6×⅜ 6×6×½	2¼ 3 3	6 8 11
4	3 to 5 6 to 10 10 to 24	2¼×5/16 3 ×⅜ 3 ×⅜	12 12 15	V
6	3 to 9 10 to 24	¾	15 18	2 2¼

All weights and dimensions marked V are variable, as are also other figures where not given.

All material for anchors steel, except Nos. 7 and 8, parts of which may be cast or malleable iron. All anchors are shipped loose and riveted or bolted to beams in the field; and in order to avoid two size holes in the same piece anchors should be so selected that holes for them and their connections may be 13/16″. The weights given above include the bolts or rivets for field connections.

Anchors Nos. 1, 2, 3, 4, and 6 are common in building construction; the split anchor bolts No. 5, with or without wedges are mostly used for bridge work and column foundations; the washer for No. 7 on outside of wall may be either a cast-iron star or a steel plate; expansion bolts No. 8 are of use in repair work to fasten channels, etc., to brick walls.

CHAPTER XVI.

STRENGTH OF CAST-IRON, WOODEN, AND STONE BEAMS.

CAST-IRON BEAMS.

OWING to the fact that the resistance of cast-iron to tension is only about one-fifth of its resistance to compression, the shapes of beams most economical for wrought-iron or steel would be wasteful if made of cast-iron. The extreme brittleness of cast iron, and its liability to flaws in casting, also render it an undesirable material for resisting transverse strain. About the only form in which cast-iron beams are now used in building construction in this country, is in the shape of lintels for supporting brick or stone walls, in places where a flat soffit is desired, and the walls are not to be plastered. Cast-iron lintels are also occasionally used over store fronts, the face of the lintel being panelled and moulded for architectural effect.

Before wrought-iron I-beams were manufactured, cast-iron beams were frequently used as the only available material, other than wood or stone. Early in the nineteenth century Mr. Eaton Hodgkinson, an English engineer, made a series of experiments with cast-iron beams from which he found that the form of cross-section of a beam which will resist the greatest transverse strain is that shown in Fig. 1, in which the bottom flange contains six times as much metal as the top flange.

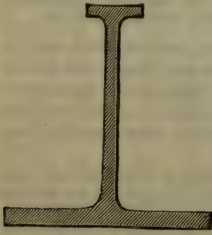


Fig. 1

When cast-iron beams are subjected to very light strains, the areas of the two flanges ought to be nearly equal. As in practice it is usual to submit beams to strains less than the ultimate load, and yet beyond a slight strain, it is found that when the flanges are as 1 to 4, we have a proportion which approximates very nearly the requirements of practice. The

thickness of the three parts—web, top flange, and bottom flange—may with advantage be made in proportion as 5, 6, and 8.

If made in this proportion, the width of the top flange will be equal to one-third of that of the bottom flange. As the result of his experiments, Mr. Hodgkinson gave the following rule for the breaking-weight at the centre for a cast-iron beam of the above form:

$$\left.\begin{matrix}\text{Breaking-load}\\ \text{in tons}\end{matrix}\right\} = \frac{\begin{pmatrix}\text{Area of bot. flange}\\ \text{in square inches}\end{pmatrix} \times \begin{pmatrix}\text{depth}\\ \text{in ins.}\end{pmatrix} \times 2.426}{\text{clear span in feet}}. \qquad (1)$$

This rule, although largely empirical, agreed very well with the few experiments that were made, and has been in general use even to the present day.

Modern structural engineers, however, use the general formula for the strength of beams, as given in Chapter XV., except that the section modulus is found by dividing the *Moment of Inertia* by the distance of the neutral axis from the bottom of the beam, and the safe tensile strength is used for the modulus of rupture.

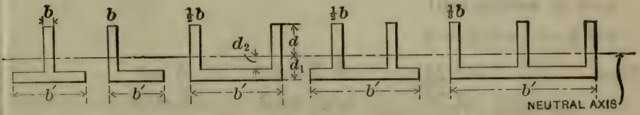
Thus the general formula for a beam supported at both ends, and with the load uniformly distributed, as given on page 502, Chapter XV., is:

Safe load in pounds$=\dfrac{2R}{3L}\times S$. As S, for cast iron, should be taken at 3,000 lbs., this formula becomes

$$\text{Safe load in pounds} = \frac{2{,}000R}{L}, \qquad (2)$$

and R for either section given below

$$= \frac{\text{Moment of Inertia}}{d_1}.$$



The moment of inertia is computed by the formula

$$I = \frac{bd^3 + b'd_1^3 - (b'-b)d_2^3}{3}, \qquad (3)$$

in which b denotes the combined thickness of the webs, and the distances d, d_1, and d_2 are measured from the neutral axis, which must pass through the *centre of gravity* of the section. The centre of gravity may be found by the method explained in Chapter VI.

This formula may be used for any of the above sections when the depth does not exceed the width, and the thickness of each web is at least equal to the thickness of the flange.

In lintels with a single web it is well to make the thickness of the web $\frac{1}{4}$ or $\frac{1}{8}$ inch greater than the thickness of the flange. For a beam of the shape shown in Fig. 1, formula (2) agrees very closely with formula (1), using a factor of safety of six.

EXAMPLE.—The following example will illustrate the application of formula (2): Compute the safe load for a cast-iron lintel having a section as shown in Fig. 2 and a clear span of 10 feet. The load to be uniformly distributed, and the thickness of the

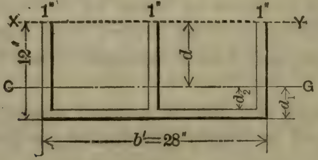


Fig. 2

metal to be one inch. The first step is to find the distance d that the centre of gravity of the section is below the top of the beam. This is found by taking the moments of the webs and flange about the line x-y, and dividing their sum by the area of the section (see page 240). Each web is 11 ins. deep and 1 in. thick, hence the area is 11 sq. inches. The moments of the three webs about x-y will then be $\quad 3 \times 11 \times 5\frac{1}{2} = 181.5$

Moment of flange about x-$y = 28 \times 11\frac{1}{2} = \underline{322}$

$$503.5$$

Area of section $= 61$.

$503.5 \div 61 = 8.25 = d$.

Then	$d = 8.25$	$d^3 = 561.5$	$b = 3$
	$d_1 = 3.75$	$d_1{}^3 = 52.7$	$b_1 = 28$
	$d_2 = 2.75$	$d_2{}^3 = 20.8$	

Next find Moment of Inertia by formula (3):

$$I = \frac{3 \times 561.5 + 28 \times 52.7 - 25 \times 20.8}{3} = 880.$$

$$R = \frac{880}{3.75} = 234.6. \quad \text{Safe load} = \frac{2000 \times 234.6}{10} = 46{,}920 \text{ lbs.}$$

or 23.4 tons.

Ends and Brackets.—When T-shaped lintels are used over a single opening, the web may be tapered towards the ends, as in Fig. 3, without affecting the strength. If the flange is more than 8 ins. wide, brackets should be cast in the middle, as at A, Fig. 3.

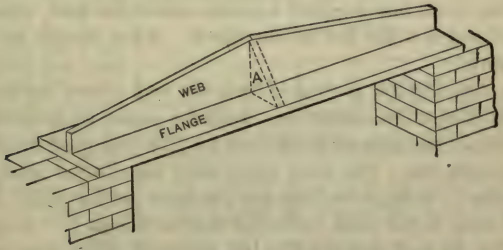


Fig. 3

When continuous lintels are used over store fronts or similar places, ends should be cast on the lintels, as in Fig. 4, and the ends of abutting lintels bolted together.

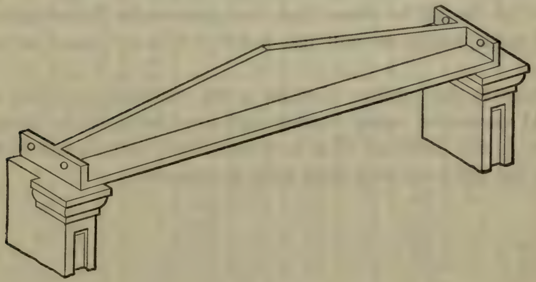


Fig. 4

All lintels with two or three webs should have solid ends connecting the webs.

Tables of Strength of Cast-iron Lintels.—The tables on the following pages have been computed in accordance with formula (2). The weight of the lintel itself should be deducted from the safe load. In using these tables it should be remem-

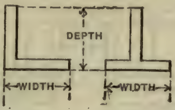
bered that the values are for loads *uniformly distributed.* If the load is concentrated at the centre, it should be multiplied by 2. If at some other point than the centre, multiply by the value on page 514 which most nearly corresponds with the position of the load. For other spans than those given multiply the distributed load by the span, and use the lintel having a coefficient C just above the product thus obtained.

EXAMPLE.—It is desired to support a 12-inch brick wall, 10 ft. high, over an opening 5 ft. 6 ins. wide, with a cast-iron lintel; 22 inches from one support a girder enters the wall, which may bring a load of 9,600 lbs. on the lintel. What should be the size of the lintel?

Ans. At 110 lbs. per cubic foot, the wall above the lintel will weigh $10 \times 5\frac{1}{2} \times 110 = 6050$ lbs. As 22 ins. is one-third of the span, we multiply the concentrated load by 1.78 (see page 514), which gives 17,088 lbs. The total equivalent distributed load is then 23,138 lbs. Multiplying this by the span we have 127,259 lbs. or 63.6 tons as the least value for the coefficient C. Looking in the table, we find that a $12'' \times 10''$ lintel, $1''$ thick, and one web, has a coefficient of 72.2, and that a $12'' \times 8'' \times 1\frac{1}{4}''$ lintel with two webs has a coefficient of 69.9. A lintel with two webs is best for a $12''$ wall, and interpolating between the values of C for $1''$ and $1\frac{1}{4}''$ thickness of the $12'' \times 8''$ lintel, we have 65.4 as the value of C for a thickness of $1\frac{1}{8}''$. This exceeds the required value by enough to more than compensate for the weight of the lintel itself, hence we will use a $12'' \times 8'' \times 1\frac{1}{8}''$ lintel with two webs.

Owing to the liability of flaws in the castings, cast-iron beams should always be tested for defects before being set in place, and if there is any doubt at all as to their safety, they should be tested up to the full load they may have to support.

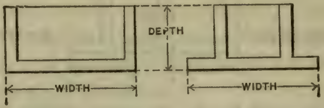
TABLE I.—SAFE DISTRIBUTED LOADS IN TONS FOR CAST-IRON LINTELS.

Lintels of  **Shapes.**

Loads include weight of lintel. Maximum tensile stress 3,000 lbs. per square inch. See remarks page 558.

Size, width by depth, ins.	Thickness of metal, ins.	Wt. per foot, lbs.	*C*, Tons.	Span in feet.							
				5	6	7	8	9	10	11	12
6×6	¾	26.3	15.9	3.18	2.65	2.27	1.98	1.76	1.59	1.44	1.32
	1	34.4	19.0	3.80	3.16	2.71	2.37	2.11	1.90	1.72	1.58
	1¼	42.0	21.5	4.30	3.58	3.07	2.68	2.39	2.15	1.95	1.79
7×6	¾	28.6	17.8	3.56	2.96	2.54	2.22	1.98	1.78	1.61	1.48
	1	37.5	21.3	4.26	3.55	3.04	2.66	2.36	2.13	1.93	1.77
	1¼	45.9	24.0	4.80	4.00	3.43	3.00	2.66	2.40	2.18	2.00
7×7	¾	31.0	22.6	4.52	3.76	3.23	2.82	2.51	2.26	2.05	1.88
	1	40.6	27.5	5.50	4.58	3.93	3.43	3.05	2.75	2.50	2.29
	1¼	49.8	31.4	6.28	5.23	4.49	3.92	3.49	3.14	2.85	2.36
8×6	¾	31.0	19.6	3.92	3.26	2.80	2.45	2.18	1.96	1.78	1.63
	1	40.6	23.4	4.68	3.90	3.34	2.92	2.60	2.34	2.12	1.95
	1¼	49.8	26.4	5.28	4.40	3.77	3.30	2.93	2.64	2.40	2.20
8×7	¾	33.3	25.0	5.00	4.16	3.57	3.12	2.77	2.50	2.27	2.08
	1	43.7	30.3	6.06	5.05	4.33	3.79	3.36	3.03	2.75	2.52
	1¼	53.7	34.8	6.96	5.80	4.97	4.35	3.86	3.48	3.16	2.90
8×8	¾	35.6	30.6	6.12	5.10	4.37	3.82	3.40	3.06	2.78	2.55
	1	46.8	37.6	7.52	6.26	5.37	4.70	4.18	3.76	3.41	3.13
	1¼	57.6	43.4	8.68	7.23	6.20	5.42	4.82	4.34	3.94	3.61
8×9	¾	38.0	36.5	7.30	6.08	5.21	4.56	4.05	3.65	3.31	3.04
	1	50.0	45.2	9.04	7.53	6.45	5.65	5.02	4.52	4.11	3.76
	1¼	61.5	52.6	10.52	8.76	7.51	6.57	5.84	5.26	4.41	4.38
12×6	¾	40.4	26.5	5.30	4.41	3.78	3.31	2.94	2.65	2.41	2.21
	1	53.1	31.6	6.32	5.26	4.51	3.95	3.51	3.16	2.87	2.63
	1¼	65.4	34.8	6.96	5.80	4.97	4.35	3.86	3.48	3.16	2.90
12×8	¾	45.0	41.7	8.34	6.95	5.95	5.21	4.63	4.17	3.79	3.48
	1	59.4	51.2	10.24	8.53	7.31	6.40	5.69	5.12	4.65	4.26
	1¼	73.2	58.5	11.70	9.75	8.35	7.31	6.50	5.85	5.32	4.87
12×10	¾	49.8	58.0	11.60	9.66	8.28	7.25	6.44	5.80	5.27	4.83
	1	65.6	72.2	14.44	12.03	10.31	9.02	8.02	7.22	6.56	6.01
	1¼	81.0	83.8	16.76	13.96	11.97	10.47	9.31	8.38	7.62	6.98
12×12	¾	54.4	75.2	15.04	12.53	10.74	9.40	8.35	7.52	6.83	6.26
	1	71.9	94.8	18.96	15.80	13.54	11.85	10.53	9.48	8.62	7.90
	1¼	88.9	111.5	22.30	18.58	15.92	13.93	12.39	11.15	10.12	9.29

TABLE I.—SAFE DISTRIBUTED LOADS IN TONS FOR CAST-IRON LINTELS—(*Continued*).

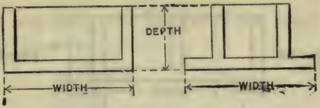
LINTELS OF 

SHAPES.

Loads include weight of lintel. Maximum tensile stress 3,000 lbs. per square inch. See remarks page 558.

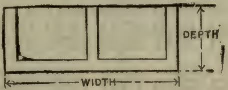
Size, width by depth, ins.	Thickness of metal, ins.	Wt. per foot, lbs.	C, tons.	Span in feet.							
				5	6	7	8	9	10	11	12
12×6	¾	52.7	31.7	6.34	5.28	4.53	3.96	3.52	3.17	2.88	2.64
	1	68.8	37.6	7.52	6.26	5.37	4.70	4.18	3.76	3.42	3.13
	1¼	84.0	43.0	8.60	7.16	6.14	5.37	4.77	4.30	3.91	3.58
12×8	¾	62.1	49.5	9.90	8.25	7.07	6.19	5.50	4.95	4.50	4.12
	1	81.3	60.9	12.18	10.15	8.70	7.61	6.76	6.09	5.53	5.07
	1¼	99.6	69.9	13.98	11.65	9.98	8.73	7.76	6.99	6.35	5.82
14×6	¾	57.4	35.5	7.10	5.91	5.07	4.43	3.94	3.55	3.22	2.96
	1	75.0	42.0	8.40	7.00	6.00	5.25	4.66	4.20	3.82	3.50
	1¼	91.8	48.0	9.60	8.00	6.85	6.00	5.33	4.80	4.36	4.00
14×8	¾	66.8	55.4	11.08	9.23	7.91	6.92	6.15	5.54	5.03	4.61
	1	87.5	68.1	13.62	11.35	9.73	8.51	7.56	6.81	6.19	5.67
	1¼	107.4	78.8	15.76	13.13	11.25	9.85	8.75	7.88	7.16	6.56
16×6	¾	62.1	39.1	7.82	6.51	5.58	4.88	4.34	3.91	3.55	3.25
	1	81.3	46.8	9.36	7.80	6.68	5.85	5.20	4.68	4.25	3.90
	1¼	99.6	52.9	10.58	8.81	7.55	6.61	5.88	5.29	4.81	4.40
16×8	¾	71.5	61.4	12.28	10.23	8.77	7.67	6.82	6.14	5.58	5.11
	1	93.8	74.6	14.92	12.43	10.65	9.32	8.29	7.46	6.78	6.21
	1¼	115.2	86.8	17.36	14.46	12.40	10.85	9.64	8.68	7.89	7.23
20×6	¾	71.5	47.2	9.44	7.86	6.74	5.90	5.24	4.72	4.29	3.93
	1	93.8	55.1	11.02	9.18	7.87	6.88	6.12	5.51	5.01	4.59
	1¼	115.2	62.0	12.40	10.33	8.85	7.75	6.88	6.20	5.63	5.16
20×8	¾	80.8	72.6	14.52	12.10	10.37	9.07	8.06	7.26	6.60	6.05
	1	106.2	89.5	17.90	14.91	12.78	11.18	9.94	8.95	8.13	7.45
	1¼	130.8	102.5	20.50	17.08	14.64	12.81	11.39	10.25	9.31	8.54
20×10	¾	90.2	100.5	20.10	16.75	14.35	12.56	11.16	10.05	9.13	8.37
	1	118.8	125.4	25.08	20.90	17.91	15.67	13.93	12.54	11.40	10.45
	1¼	146.5	146.8	29.36	24.46	20.97	18.35	16.31	14.68	13.34	12.23
20×12	¾	99.6	122.6	24.52	20.43	17.51	15.32	13.62	12.26	11.14	10.21
	1	131.3	158.0	31.60	26.33	22.57	19.75	17.55	15.80	14.36	13.16
	1¼	162.1	189.5	37.90	31.58	27.07	23.68	21.05	18.95	17.22	15.79
24×8	¾	90.2	83.4	16.68	13.90	11.91	10.42	9.26	8.34	7.58	6.95
	1	118.8	102.4	20.48	17.06	14.63	12.80	11.37	10.24	9.31	8.53
	1¼	146.5	117.0	23.40	19.50	16.71	14.62	13.00	11.70	10.63	9.75

TABLE I.—SAFE DISTRIBUTED LOADS IN TONS FOR CAST-IRON LINTELS—(*Continued*).

LINTELS OF  SHAPES.

Loads include weight of lintels. Maximum tensile stress 3,000 lbs. per square inch. See remarks page 558.

Size, width by depth, ins.	Thickness of metal, ins.	Wt. per foot, lbs.	*C*, tons.	Span in feet.							
				5	6	7	8	9	10	11	12
24×10	¾	99.6	116.0	23.20	19.33	16.57	14.50	12.88	11.60	10.54	9.66
	1	131.3	144.4	28.88	24.06	20.63	18.05	16.04	14.44	13.12	12.03
	1¼	162.1	167.6	33.52	27.93	23.94	20.95	18.62	16.76	15.23	13.96
24×12	¾	109.0	150.4	30.08	25.06	21.48	18.80	16.71	15.04	13.67	12.53
	1	143.8	189.6	37.92	31.60	27.08	23.70	21.06	18.96	17.23	15.80
	1¼	177.7	223.0	44.60	37.16	31.85	27.87	24.77	22.30	20.27	18.58
28×8	¾	99.6	95.5	19.10	15.25	13.64	11.93	10.61	9.55	8.68	7.98
	1	131.3	115.0	23.00	19.16	16.43	14.37	12.77	11.50	10.45	9.58
	1¼	162.1	130.5	26.10	21.75	18.64	16.31	14.50	13.05	11.86	10.87
28×10	¾	109.0	140.5	28.10	23.41	20.07	17.56	15.61	14.05	12.77	11.70
	1	143 8	164.8	32.96	27.46	23.54	20.60	18.31	16.48	14.98	13.73
	1¼	177.7	192.0	38.40	32.00	27.43	24.00	21.33	19.20	17.45	16.00
28×12	¾	118.3	171.4	34.28	28.56	24.48	21.82	19.04	17.14	15.58	14.28
	1	156.3	216.1	43.22	36.01	30.87	27.01	24.01	21.61	19.64	18.00
	1¼	193.3	256.7	51.34	42.78	36.67	32.08	28.52	25.67	23.33	21.39

LINTELS OF  SHAPES.

16×6	¾	74.4	43.3	8.66	7.21	6.18	5.41	4.81	4.33	3.93	3.60
	1	96.9	52.4	10.48	8.73	7.48	6.55	5.82	5.24	4.76	4.36
	1¼	118.1	59.3	11.86	9.88	8.47	7.41	6.59	5.93	5.39	4.86
16×8	¾	88.5	68.1	13.62	11.35	9.73	8.51	7.56	6.81	6.19	5.67
	1	115.6	83.9	16.75	13.98	11.98	10.48	9.32	8.39	7.62	6.99
	1¼	141.6	97.0	19.40	16.16	13.85	12.12	10.77	9.70	88.1	8.08
20×8	¾	97.8	80.2	16.04	13.36	11.45	10.02	8.91	8.02	7.29	6.68
	1	128.1	98.7	19.74	16.45	14.10	12.33	10.96	9.87	8.97	8.22
	1¼	157.2	113.9	22.78	18.98	16.27	14.23	12.65	11.39	10.35	9.49

TABLE I.—SAFE DISTRIBUTED LOADS IN TONS FOR CAST-IRON LINTELS—(*Continued*).

LINTELS OF 

SHAPES.

Loads include weight of lintel. Maximum tensile stress 3,000 lbs. per square inch. See remarks page 558.

Size, width by depth, ins.	Thickness of metal, ins.	Wt. per foot, lbs.	C, tons.	Span in feet.							
				5	6	7	8	9	10	11	12
20×10	¾	111.9	112.0	22.40	18.66	16.00	14.00	12.44	11.20	10.18	9.33
	1	146.9	139.7	27.94	23.28	19.95	17.46	15.52	13.97	12.70	11.64
	1¼	180.7	163.5	32.70	27.25	23.35	20.43	18.16	16.35	14.86	13.62
20×12	¾	126.0	146.7	29.34	24.45	20.95	18.33	16.30	14.67	13.33	12.22
	1	165.6	184.8	36.96	30.80	26.40	23.10	20.53	18.48	16.80	15.40
	1¼	204.1	218.8	43.76	36.46	31.25	27.35	24.31	21.88	19.89	18.33
24×8	¾	107.2	91.9	18.38	15.31	13.12	11.49	10.21	9.19	8.35	7.66
	1	140.6	112.8	22.56	18.80	16.11	14.10	12.53	11.28	10.25	9.40
	1¼	172.6	130.2	26.04	21.70	18.57	16.27	14.47	13.02	11.83	10.85
24×10	¾	121.3	127.8	25.56	21.30	18.25	15.97	14.20	12.78	11.61	10.65
	1	159.4	159.5	31.90	26.58	22.78	19.94	17.72	15.95	14.50	13.29
	1¼	196.3	183.6	36.72	30.60	26.23	22.95	20.40	18.36	16.69	15.30
24×12	¾	135.3	166.6	33.32	27.76	23.80	20.82	18.51	16.66	15.14	13.88
	1	178.1	209.3	41.86	34.88	29.90	26.16	23.25	20.93	19.02	17.44
	1¼	219.7	247.7	49.54	41.28	35.39	30.96	27.52	24.77	22.51	20.64
28×10	¾	130.7	141.4	28.28	23.57	20.20	17.67	15.71	14.14	12.85	11.78
	1	171.9	177.4	35.48	29.57	25.34	22.17	19.71	17.74	16.12	14.78
	1¼	211.9	207.8	41.56	34.63	29.68	25.97	23.09	20.78	18.89	17.31
28×12	¾	144.7	186.0	37.20	31.00	26.57	23.25	20.66	18.60	16.91	15.50
	1	190.6	234.6	46.92	39.10	33.51	29.32	26.06	23.46	21.32	19.55
	1¼	235.3	277.9	55.58	46.31	39.70	34.74	30.88	27.79	25.26	23.16

Strength of Wooden Beams.

Wooden beams are almost invariably square or rectangular shaped timbers, and we shall therefore consider only that shape in the following rules and formulas.

For beams with a rectangular cross-section, we can simplify our formulas for strength by substituting for the moment of inertia its value, viz., $\frac{b \times d^3}{12}$, where b = breadth of beam and d its depth.

Then, substituting this value in the general formulas for beams, we have for rectangular beams *of any material* the following formulas:—

Beams fixed at one end, and loaded at the other (Fig. 5).

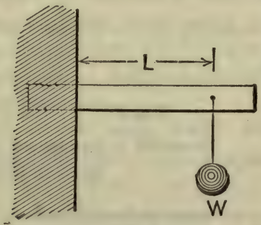


Fig. 5

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{4 \times \text{length in feet}}, \quad (4)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A}. \quad (5)$$

Beams fixed at one end, and loaded with uniformly distributed load (Fig. 6).

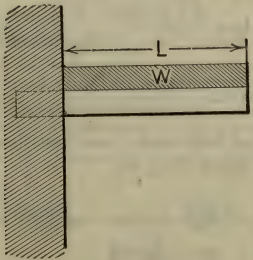


Fig. 6

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{2 \times \text{length in feet}}, \quad (6)$$

or

$$\text{Breadth in inches} = \frac{2 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A}. \quad (7)$$

* For value of A, see Table II.

Beams supported at both ends, loaded at middle (Fig. 7).

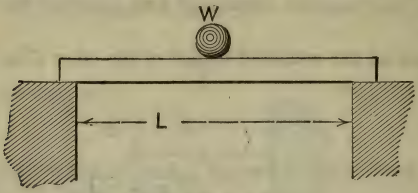


Fig. 7

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A\,*}{\text{span in feet}}, \quad (8)$$

or

$$\text{Breadth in inches} = \frac{\text{span in feet} \times \text{load}}{\text{square of depth} \times A}. \quad (9)$$

Beams supported at both ends, load uniformly distributed (Fig. 8).

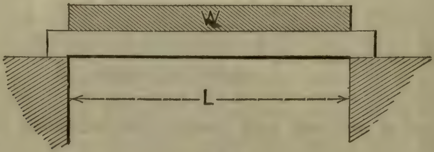


Fig. 8

$$\text{Safe load in pounds} = \frac{2 \times \text{breadth} \times \text{square of depth} \times A\,*}{\text{span in feet}}, \quad (10)$$

or

$$\text{Breadth in inches} = \frac{\text{span in feet} \times \text{load}}{2 \times \text{square of depth} \times A}. \quad (11)$$

Beams supported at both ends, load uniformly distributed over only a portion of the span (Fig. 9).

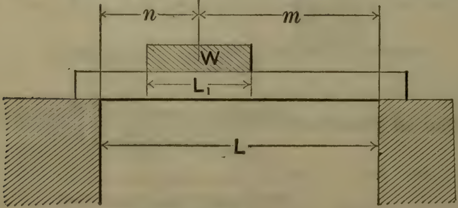


Fig. 9

* For value of A, see Table II.

In this case the dimensions of the beam required to carry the load can be accurately determined only by computing the bending-moment, as explained in Chapter IX. and substituting the value thus found in formula (16), following. If, however, the length L_1 is very short in comparison with L, then the load may be considered as concentrated at its centre, and the breadth of the beam may be found by formula (9), if the load is at the centre of the span, or by formula (13), if it is at one side of the centre. The error will be on the safe side.

Beams supported at both ends, loaded with concentrated load NOT AT CENTRE (Fig. 10).

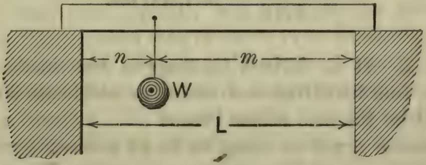


Fig. 10

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{sq. of depth} \times \text{span} \times A^*}{4 \times m \times n}, \quad (12)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load} \times m \times n}{\text{square of depth} \times \text{span} \times A}. \quad (13)$$

Beams supported at both ends, and loaded with W *pounds at a distance* m *from each end* (Fig. 11).

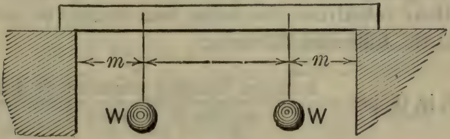


Fig. 11

$$\begin{array}{c}\text{Safe load } W \text{ in pounds}\\ \text{at each point}\end{array} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{4 \times m}, \quad (14)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load at one point} \times m}{\text{sq. of depth} \times A}. \quad (15)$$

NOTE.—In the last two cases the lengths denoted by m and n should be taken in feet, the same as the spans.

---

* For value of A, see Table II.

Application of the foregoing formulas.

Example 1.—What load will an 8 inch by 12 inch hard pine beam, securely fastened into a brick wall at one end, sustain with safety, 6 feet out from the wall?

Ans. Safe load in pounds (Formula 4) equals

$$\frac{8 \times 144 \times 100}{4 \times 6} = 4{,}800 \text{ lbs.}$$

Example 2.—It is desired to suspend two loads of 10,000 pounds each, four feet from each end of an oak beam 20 feet long. What should be the size of the beam?

Ans. Assume depth of beam to be 14 inches; then (Formula 15) breadth $= \frac{4 \times 10{,}000 \times 4}{196 \times 75} = 11$ inches, nearly; therefore the beam should be 11×14 inches.

To find the size of beam (supported at both ends) to support several concentrated loads, or a distributed load and one or more concentrated loads.—The correct method of finding the least size of beam that will safely support a combination of loads, is to first find the maximum bending-moment, as explained in Chapter IX., and then substitute the value thus found for the bending-moment in the following formula:

$$\text{Breadth in inches} = \frac{4 \times \text{max. bending moment, ft. lbs.}}{\text{square of depth} \times A}. \qquad (16)$$

An example of this kind for steel beams is worked on pages 504–506.

A shorter and easier method is to find the *equivalent* distributed load for each concentrated load, and then find the size of beam required to support the total equivalent distributed load thus found. The equivalent distributed load for concentrated loads applied at different proportions of the span from either end, may be obtained by multiplying the concentrated load by the following factors:

For concentrated	load	applied	at centre of span,	multiply by	2
"	"	"	" at 1/3d	the span, by	1.78
"	"	"	" at 1/4th	" " by	1.5
"	"	"	" at 1/5th	" " by	1.28
"	"	"	" at 1/6th	" " by	$1\frac{1}{9}$
"	"	"	" at 1/7th	" " by	.98
"	"	"	" at 1/8th	" " by	$\frac{7}{8}$
"	"	"	" at 1/9th	" " by	.79
"	"	"	" at 1/10th	" " by	.72

Thus a concentrated load of 900 lbs. applied at one-sixth of the span from one support, will produce the same bending-moment as a distributed load of $900 \times 1\frac{1}{9}$ or 1,000 lbs.

The above method of finding the size of beam for a combination of several loads, will give a *larger beam* than the correct method, by formula (16), for the reason that the maximum bending-moment will not be equal to the sum of the individual bending-moments, hence when there are several heavy loads to be supported, it will be economy to compute the maximum bending-moment by the graphic method explained in Chapter IX.

EXAMPLE 3.—The girder *G*, Fig. 12, supports the rafters of a flat roof, and also three heavy beams, *A*, *B*, *C*, blocked up above the roof and supporting a large tank filled with water. The timber is to be longleaf yellow pine. The weight of the roof and allowance for snow will be 7,500 lbs. Each of the beams *A*, *B*, and *C* will impose a load on the girder due to the weight of the tank and its contents of 3,000 lbs. What should be the size of the girder?

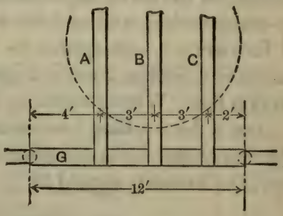


Fig. 12

Ans. The roof load may be considered as uniformly distributed. The load from beam *A* is applied 1/3d the span from one end; the load from *B* 5/12ths the span from the other end, and the load from *C* 1/6th the span. The fraction 5/12ths is half way between 1/2 and 1/3d; hence the load from *B* should be multiplied by 1.89. Multiplying the concentrated loads by their proper factors, we find the *equivalent* distributed load to be as follows:

Roof load, distributed	=	7,500
Load from *A*, $3{,}000\times1.78$	=	5,340
Load from *B*, $3{,}000\times1.89$	=	5,670
Load from *C*, $3{,}000\times1\frac{1}{9}$	=	3,333

Equivalent distributed load = 21,843 lbs.

Assuming 14 ins. as the depth of the beam, and using formula (11), we have

$$\text{Breadth in ins.} = \frac{12\times21{,}843}{2\times196\times100} = 6.7 \text{ ins.}$$

Assuming 12 ins. for the depth, we obtain 9.1 ins. for the breadth, hence the girder must be $10''\times12''$, or $7''\times14''$.

Strut Beams and Tie Beams.

A "strut beam" is a beam that is subject both to a transverse strain and to a compressive stress.

A "tie beam" is one that is subject to direct tension in addition to the transverse strain.

To find the strength of either, first find the size of beam required to resist the transverse strain, and then the size of timber, of the same depth as the beam, to resist the direct tension or compression, and add the two breadths together.

Example 4.—A spruce tie beam 10 feet long between joints sustains a ceiling load of 2,000 lbs. and a direct tensile stress of 40,000 lbs. What should be the dimensions of the beam?

Ans. As a ceiling load is uniformly distributed we determine the size of the beam by formula (11). Assuming the depth as 8 ins., the breadth should be

$$\frac{10\times2{,}000}{2\times64\times70} \text{ or } 2\tfrac{3}{8} \text{ ins. nearly.}$$

The resistance of spruce to tension (see Table I., Chapter XI.) is 1,600 lbs. 40,000 divided by 1,600 = 25 sq. ins., which is equivalent to $3\frac{1}{8}\times8$ ins.; therefore it will require a beam $5\frac{1}{2}''\times 8''$ to resist both the transverse strain and the direct tension. If the tie beam is cut in any way so as to reduce the section (except over a support) the dimensions must be increased accordingly.

Example 5.—A strut beam of white pine 10 feet long supports a distributed roof load of 6,000 lbs., and is also subject to a

direct compression of 48,000 lbs. What should be the size of the beam?

Ans. Assuming 12 inches for the depth, we find the breadth for the transverse load by formula 11

$$\text{Breadth} = \frac{10 \times 6{,}000}{2 \times 144 \times 60} = 3\tfrac{1}{2} \text{ ins. nearly.}$$

Looking in the table giving the strength of white pine posts, Chapter XIV., we find that an 8×12 post 10 feet long will support 51,450 lbs., or a little more than our compressive stress. Hence it will require an 8×12 beam to resist the compressive stress and a beam $3\frac{1}{2} \times 12$ to resist the transverse load. We should therefore make the beam 12×12 ins. to resist them both.

Values of the Constant *A*.

The letter *A* in formulas 4–16 denotes the safe load for a unit beam one inch square and one foot span, loaded at the centre. This is also one-eighteenth of the modulus of rupture or fibre stress for safe loads. The following are the values of *A*, which are obtained by dividing the moduli of rupture in Chap. XV. by 18.

TABLE II.—VALUES OF *A*.—CO-EFFICIENT FOR BEAMS.

Material.	*A* lbs.	Material.	*A* lbs.
Cast iron	308	Pine, white, Western	65
Wrought iron	666	" Texas yellow	90
Steel	888	Spruce	70
		Whitewood (poplar)	65
American woods:		Redwood (California)	60
Chestnut	60	Bluestone flagging (Hudson River)	25
Hemlock	55		
Oak, white	75	Granite, average	17
Pine, Georgia yellow	100	Limestone	14
" Oregon	90	Marble	17
" red or Norway	70	Sandstone	8 to 11
" white, Eastern	60	Slate	50

These values for the co-efficient *A* are one-third of the breaking-weight of timbers of the same size and quality as that used in first-class buildings. This is a sufficient allowance for timbers in roof trusses, and beams which do not have to carry a more severe load than that of a dwelling-house floor, and small halls, etc. Where there is likely to be very much vibration, as in the floor of a mill, or a gymnasium floor, or floors of large public halls,

the author recommends that only four-fifths of the above values for A be used.

For beams supporting permanent loads, such as masonry, or water-tanks, the safe load should be reduced ten per cent., as such loads are not usually overestimated.

The values for stones are based on a factor of safety of six.

For comparative values of A, as given in Building Laws, see page 573.

Relative Strength of Rectangular Beams.

From an inspection of the foregoing formulas it will be found that the relative strength of rectangular beams in different cases is as follows:

Beam supported at both ends, and loaded with a uniformly distributed load 1

Beams supported at both ends:

Load uniformly distributed		1
Concentrated load at centre		$\frac{1}{2}$
" "	one-third the span	$\frac{9}{16}$
" "	one-fourth "	$\frac{2}{3}$
" "	one-fifth "	$\frac{25}{32}$
" "	one-sixth "	$\frac{9}{10}$
" "	one-seventh "	$\frac{49}{48}$
" "	one-eighth "	$\frac{8}{7}$
" "	one-ninth "	$\frac{81}{64}$
" "	one-tenth "	$\frac{25}{18}$

Beam fixed at one end, and loaded with a uniformly distributed load.. $\frac{1}{4}$

Beam fixed at one end, and loaded at the other............. $\frac{1}{8}$

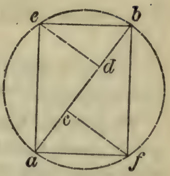
Also the following can be shown to be true:

Beam firmly fixed at both ends, and loaded at the centre.... 1

Beam fixed at both ends, and loaded with distributed load...$1\frac{1}{2}$

These facts are also true of a uniform beam of any form of cross-section.

When a square beam is supported on its edge, instead of on its side—that is, has its diagonal vertical—it will bear about seven-tenths as great a breaking-load.



The strongest beam which can be cut out of a round log is one in which the breadth is to the depth as 5 to 7, very nearly, and can be found graphically, as shown in margin. Draw any diagonal, as *ab*, and divide it into three equal parts by the points *c* and *d*; from these points draw perpendicular lines, and connect the points *e* and *f* with *a* and *b*, as shown.

Cylindrical Beams.—A cylindrical beam is only $\frac{10}{17}$ as strong as a square beam whose side is equal to the diameter of the circle. Hence, to find the load for a cylindrical beam, first find the proper load for the corresponding square beam, and then divide it by 1.7.

The bearing of the ends of a beam on a wall beyond a certain amount does not strengthen the beam any. In general, a beam should have a bearing of four inches, and if the beam be very long, the bearing should be 6 ins.

Weight of the Beam itself to be taken into Account.—The formulas we have given for the strength of beams do not take into account the weight of the beam itself, and hence the safe load of the formulas includes both the external load and the weight of the material in the beam. In small wooden beams, the weight of the beam is generally so small, compared with the external load, that it need not be taken into account. But in larger wooden beams, and in metal and stone beams, the weight of the beam should be subtracted from the safe load if the load is distributed; and if the load is applied at the centre, one-half the weight of the beam should be subtracted.

The weight per cubic foot for different kinds of timber may be found in the table giving the *Weight of Substances*, Part III.

Explanation of Tables III.-VII.

Tables for the strength of yellow and white pine, Oregon pine, spruce, and oak beams, are given on the following pages for beams one inch wide. These tables were computed by the author from the rules and coefficients given in this chapter, and are believed to be perfectly reliable when used in accordance with the explanations.

To find the strength of a given beam of any other breadth, it is only necessary to multiply the strength given in the table by the breadth of the given beam.

Exampe 6.—What is the safe distributed load for a yellow-pine

beam, supported at both ends, 8 inches by 12 inches, 20 feet clear span?

Ans. From Table III., safe load for one inch thickness is 1,440 pounds; $1,440 \times 8 = 11,520$ pounds, safe load for beam.

To find the size of a beam that will support a given load with a given span, find the safe load for a beam of an assumed depth one inch wide, and divide the given load by this strength.

EXAMPLE 7.—What size spruce beam will be required to carry a distributed load of 8,640 pounds for a clear span of 18 feet?

Ans. From the table for spruce beams, we find that a beam 14 inches deep and 1 inch thick, 18 feet span, will support 1,524 pounds; and dividing the load, 8,640 pounds, by 1,524, we have $5\frac{1}{2}$ for the breadth of the beam in inches: hence the beam should be 6×14 inches, to carry a distributed load of 8,640 pounds with a span of 18 feet.

To find the safe centre load of a given beam, first find the safe distributed load as in Example 6, and divide by two.

To find the safe load when concentrated at some point other than the centre, first find the safe distributed load for the given span, and divide by the factors given on page 567.

To find the size of beam to support a given concentrated load, multiply the given load by the factor corresponding with the position of the load as given on page 567, and then proceed as in Example 7.

If in doubt as to the application of the tables, in special cases, it will be safer to use the appropriate formula, as given on pages 563 and 565. The formulas and tables should always give the same result.

To use the tables for beams that run less than the nominal dimensions. In many localities floor joists as carried in stock are more or less scant of the nominal dimensions, and for such joists a reduction in the safe load must be made to correspond to the reduction in size. For beams $\frac{1}{4}$ inch scant in *both* dimensions the safe load may be obtained by multiplying the safe load as given in the table by the following factors:

For beams $1\frac{3}{4}'' \times 5\frac{3}{4}''$	by 1.6	For beams $1\frac{3}{4}'' \times 11\frac{3}{4}''$	by 1.67
$2\frac{3}{4}'' \times 5\frac{3}{4}''$	" 2.52	$2\frac{3}{4}'' \times 11\frac{3}{4}''$	" 2.63
$1\frac{3}{4}'' \times 6\frac{3}{4}''$	" $1\frac{5}{8}$	$1\frac{3}{4}'' \times 13\frac{3}{4}''$	" 1.68
$2\frac{3}{4}'' \times 6\frac{3}{4}''$	" 2.55	$2\frac{3}{4}'' \times 13\frac{3}{4}''$	" 2,65
$1\frac{3}{4}'' \times 7\frac{3}{4}''$	" 1.64	$1\frac{3}{4}'' \times 14\frac{3}{4}''$	" 1.69
$2\frac{3}{4}'' \times 7\frac{3}{4}''$	" 2.58	$2\frac{3}{4}'' \times 14\frac{3}{4}''$	" 2.66
$1\frac{3}{4}'' \times 9\frac{3}{4}''$	" 1.66	$1\frac{3}{4}'' \times 15\frac{3}{4}''$	" 1.7
$2\frac{3}{4}'' \times 9\frac{3}{4}''$	" 2.61	$2\frac{3}{4}'' \times 15\frac{3}{4}''$	" 2.66

EXAMPLE.—What is the safe load for a $2\frac{3}{4}'' \times 13\frac{3}{4}''$ Oregon pine beam, 20 feet span?

Ans. From Table IV. we find the safe load for a 1×14 beam to be 1,764 lbs. Multiplying this by 2.65 we have 4,674 lbs. as the safe distributed load for a beam $2\frac{3}{4} \times 13\frac{3}{4}$ ins. For a beam full 3×14 ins. the safe load is 5,292 lbs.

Stone Beams.—The same formulas apply to stone as to wooden beams, but the values of the co-efficient A are only from one-sixth to one-tenth of breaking-loads. Sandstone beams should never be subjected to any very heavy loads; but, where used as lintels, the stone should be relieved by iron beams or brick arches back of the stone.

Comparison of the values of A (for the transverse strength of wooden beams) given in Building Laws with those of Table II. (A as used by the author is $\frac{1}{18}$ of the fibre stress, and $\frac{1}{2}$ of the constant C, given in the Building Laws of Buffalo and Chicago).

Kind of wood.	Maximum working values for A.						
	Buffalo.	Boston.	Chicago.	Denver.	New York.	*	Kidder.
Yellow Pine.....	100	69	80	100	66	$66\frac{2}{3}$	100
Oregon Pine.....	.....	.....	.....	90	.....	61	90
White Pine......	60	.....	50	75	$44\frac{1}{2}$	40	60–65
Spruce..........	.....	42	.....	.....	$44\frac{1}{2}$	40	70
Oak (White)....	75	$55\frac{1}{2}$	60	90	$55\frac{1}{2}$	$55\frac{1}{2}$	75
Chestnut........	.....	.....	.....	.....	$44\frac{1}{2}$	$44\frac{1}{2}$	60
Hemlock.......	60	.....	.....	.....	$33\frac{1}{3}$	$33\frac{1}{3}$	55

* The values in this column were recommended by the Committee on Strength of Bridge and Trestle Timbers of the Association of Railway Superintendents of Bridges and Buildings, in 1895, and are supposed to give *a factor of safety of six.* With a factor of safety of *four* they agree very closely with the values recommended and used by the author for ordinary floor beams and girders.

TABLE III.—HARD-PINE BEAMS.

Table of safe quiescent loads for horizontal rectangular beams of Georgia yellow pine, one inch broad, supported at both ends, load *uniformly distributed*. For *concentrated* load at centre *divide by two*. For *permanent* loads (such as masonry) reduce by 10 per cent.

Depth of beam.	Span in feet.												
	6	8	10	12	14	15	16	18	20	22	24	25	27
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
6	1,200	900	720	600	514	480							
7	1,633	1,225	980	816	700	653	612						
8	2,133	1,600	1,280	1,066	914	853	800						
9	2,700	2,025	1,620	1,350	1,157	1,080	1,012	900					
10	3,333	2,500	2,000	1,666	1,428	1,333	1,250	1,111	1,000				
12	4,800	3,600	2,880	2,400	2,056	1,920	1,800	1,600	1,440				
14	6,533	4,900	3,920	3,266	2,800	2,613	2,450	2,177	1,960	1,782	1,633	1,568	1,450
15	7,500	5,633	4,500	3,750	3,214	3,000	2,816	2,500	2,250	2,045	1,875	1,800	1,666
16	8,533	6,400	5,120	4,266	3,656	3,412	3,200	2,844	2,560	2,327	2,133	2,048	1,896

Loads above and to the right of heavy line will crack plastered ceilings.

TABLE IV.—OREGON-PINE (DOUGLAS FIR) BEAMS.

Table of safe quiescent loads for horizontal rectangular beams of Oregon pine (Douglas fir), one inch broad, supported at both ends, load *uniformly distributed*. For *concentrated* load at centre *divide by two*. For *permanent* loads (such as masonry) reduce by 10 per cent.

Depth of beam.	Span in feet.												
	6	8	10	12	14	15	16	18	20	22	24	25	27
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
6	1,080	810	648	540									
7	1,470	1,102	882	735									
8	1,920	1,440	1,152	960	823	768	720						
9	2,430	1,822	1,458	1,215	1,041	972	911	810					
10	3,000	2,250	1,800	1,500	1,286	1,200	1,125	1,000	900				
12	4,320	3,240	2,592	2,160	1,851	1,728	1,620	1,440	1,296	1,178	1,080	1,036	
14	5,880	4,410	3,528	2,940	2,520	2,352	2,205	1,960	1,764	1,604	1,470	1,411	1,306
15	6,750	5,062	4,050	3,375	2,892	2,700	2,531	2,250	2,025	1,841	1,687	1,620	1,500
16	7,680	5,760	4,608	3,840	3,291	3,072	2,880	2,560	2,304	2,094	1,920	1,843	1,707

Loads above and to the right of heavy line will crack plastered ceilings.

TABLE V.—WHITE OAK BEAMS.

Table of safe quiescent loads for horizontal rectangular beams, one inch broad, supported at both ends, load *uniformly distributed*. For *concentrated* load at centre *divide by two*. For *permanent* loads (such as masonry) reduce by 10 per cent.

Depth of beam.	Span in feet.												
	6	8	10	12	14	15	16	17	18	20	22	24	25
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
6	900	675	540	450	386	360							
7	1,225	919	735	612	525	490	459						
8	1,600	1,200	960	800	685	640	600	565					
9	2,025	1,519	1,215	1,012	868	801	759	715	675				
10	2,500	1,875	1,500	1,250	1,071	1,000	937	882	833	750			
12	3,600	2,700	2,160	1,800	1,544	1,440	1,350	1,270	1,200	1,080			
14	4,900	3,675	2,940	2,450	2,100	1,960	1,837	1,729	1,633	1,470	1,336		
15	5,625	4,218	3,375	2,812	2,410	2,250	2.109	1,985	1,875	1,687	1 ,534	1,406	1,350
16	6,400	4,800	3,840	3,200	2,742	2,560	2,400	2,260	2,133	1,920	1,745	1,600	1,536

Loads above and to the right of heavy line will crack plastered ceilings.

TABLE VI.—SPRUCE BEAMS.

Table of safe quiescent loads for horizontal rectangular beams, one inch broad, supported at both ends, load *uniformly distributed*. For *concentrated* load at centre *divide by two*. For *permanent* loads (such as masonry) reduce by 10 per cent.

Depth of beam.	Span in feet.												
	6	**8**	**10**	**12**	**14**	**15**	**16**	**17**	**18**	**20**	**22**	**24**	**25**
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
6	840	630	504	420	360	336							
7	1,143	857	686	572	490	457	428						
8	1,493	1,120	896	746	640	597	560	527					
9	1,890	1,417	1,134	945	810	756	708	667	630				
10	2,333	1,750	1,400	1,166	1,000	933	875	824	777	700			
12	3,360	2,520	2,016	1,680	1,440	1,344	1,260	1,186	1,120	1,018			
14	4,573	3,430	2,744	2,286	1,960	1,828	1,715	1,614	1,524	1,372	1,247	1,143	1,097
15	5,250	3,937	3,150	2,625	1,875	2,100	1,968	1,853	1,750	1,575	1,431	1,312	1,260
16	5,973	4,480	3,584	2,986	2,540	2,388	2,240	2,108	1,991	1,792	1,629	1,493	1,433

Loads above and to the right of heavy line will crack plastered ceilings.

TABLE VII.—WHITE-PINE (OR COMMON SOFT PINE) BEAMS.

Table of safe quiescent loads for horizontal rectangular beams, one inch broad, supported at both ends, load *uniformly distributed*. For *concentrated* load at centre *divide by two*. For *permanent* loads (such as masonry) reduce by 10 per cent.

Depth of beam.	Span in feet.												
	6	**8**	**10**	**12**	**14**	**15**	**16**	**17**	**18**	**20**	**22**	**24**	**25**
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
6	720	540	432	360	308								
7	980	735	588	490	420	392							
8	1,280	960	768	640	548	512	480						
9	1,620	1,215	972	810	694	648	607	572					
10	2,000	1,500	1,200	1,000	857	800	750	705	666				
12	2,880	2,160	1,728	1,440	1,234	1,152	1,080	1,016	960	864			
14	3,920	2,940	2,352	1,960	1,680	1,568	1,470	1,383	1,306	1,176	1,069	980	940
15	4,500	3,375	2,700	2,250	1,928	1,800	1,687	1,588	1,500	1,350	1,227	1,125	1,080
16	5,120	3,840	3,072	2,560	2,192	2,048	1,920	1,807	1,706	1,536	1,396	1,280	1,230

Loads above and to the right of heavy line will crack plastered ceilings.

CHAPTER XVII.

STRENGTH OF BUILT-UP WOODEN BEAMS, FLITCH-PLATES, AND TRUSSED GIRDERS.

Built-up Wooden Beams.—Wooden beams or girders built up of planks, spiked or bolted together *side by side,* will generally be somewhat stronger than a solid beam of the same dimensions, because the planks will be better seasoned and more free from check cracks and other defects. For beams or girders 10 ins. or less in depth spikes will usually be sufficient for binding the planks together, but for deeper beams bolts should be used in addition to the spikes, to prevent the planks from separating and the outer planks from warping or curling away from the others.

Two bolts should be placed at each end of the beam and about four feet apart between.

When beams are built up in this way *each plank should be the full length of the beam,* or, in the case of a continuous beam, the planks should break joint over the supports. Built beams should always be set with the planks on edge, and *never flatways.*

Compound Wooden Girders.—It is often desirable to use a wooden girder for a longer span or greater load than would be safe for the deepest single beam that can be obtained, or for a beam built up of planks. In such cases compound wooden beams may be used.

By a compound wooden beam or girder is meant a beam built up by placing two or more single beams one *on top* of the other, with the view of having them act as a single beam having the depth of the combined beams.

Thus if two 10×10-inch beams were placed one on top of the other, and the upper one loaded at the centre, the beams would act as two separate beams (Fig. 1) and their combined strength would be no greater than if the two beams were placed side by side. If, however, the two beams can be joined so that the fibres of the lower beam will be extended as much as would be the case in a single beam of the same depth or in other words, so that the

two beams will not slip on each other, the compound beam will have four times the strength of the single beam.

Various attempts have been made to join beams thus placed so as to prevent the two parts slipping on each other, but until within a few years there has been no experimental data to show how far such methods accomplish their object.

During the years 1896–7, however, Prof. Edgar Kidwell, of the Michigan College of Mines, made quite an extended series of tests

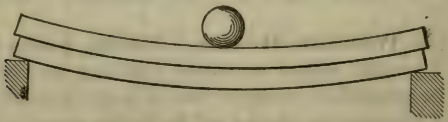


Fig. 1

of the efficiency of compound beams of different patterns, and from these tests much valuable data has been obtained. A full description of the tests accompanied by the conclusions of the author, and rules and data for proportioning the bolts and keys, of keyed beams, is published in Vol. XXVII., "Transactions of the American Institute of Mining Engineers."

Probably the most common form of compound beam, as used in American building construction, is that shown in Fig. 2,

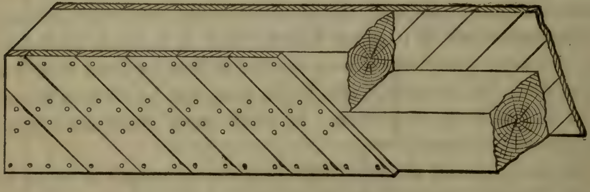


Fig. 2

diagonal boards in opposite directions being nailed to each side of the two timbers to prevent their slipping on each other. Mr. T. M. Clark, in his "Building Superintendence," advocates this as one of the best forms of compound beams, and places its efficiency at about 95 per cent. of a solid beam of the same depth.

Prof. Kidwell made nine tests of this style of beam, six having a ratio of span to depth of beam as 12 to 1, and three as 24 to 1. The shorter beams gave an average efficiency without much

variation, of 71.4 per cent., and the longer beams an efficiency of 80.7.

It was found that the beams failed by the splitting of the diagonal pieces or the drawing of the nails—"in every case, long before the beam broke, the struts split open or the nails were drawn partly out, or bent over in the wood, thereby permitting the component beams to slide on each other. It was found that no amount of nailing could prevent this."

When built with diagonal boards $1\frac{1}{4}$ inches thick, nailed with 10 d's as in Fig. 2, the working strength of such a beam may be taken at 65 per cent. of the strength of a solid beam of the same depth, and of a breadth equal to the breadth of the timbers. The deflection of the beam, however, *will be about double* that of a solid beam of the same size, and on that account this style of beam is not to be recommended for supporting floors with plastered ceilings or carrying plastered partitions.

Keyed Beams. — Prof. Kidwell also tested several styles of keyed beams, with the result that a compound beam keyed and bolted together, as shown in Fig. 3, was found to be the most efficient form that it is practicable to build.

It was found that with oak keys it was possible to obtain an efficiency for spruce beams

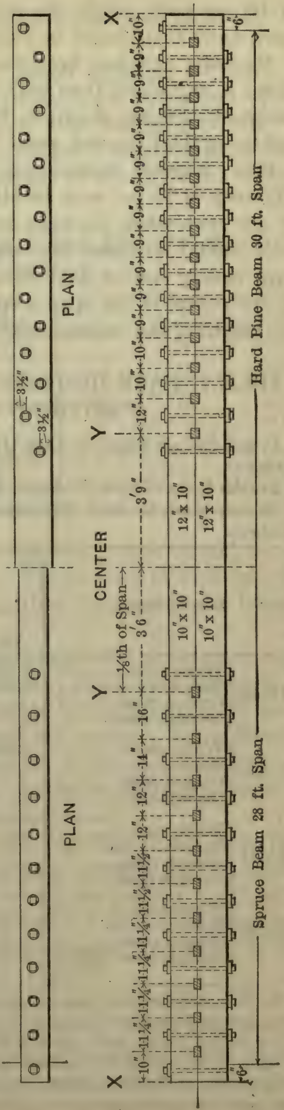


Fig. 3.—Compound Beams.

of 95 per cent., while the deflection varied from 20 to 25 per cent. more than would be expected in a solid beam.

By using cast-iron keys the deflection was found to be but little, if any more, than with a solid beam. The keys must be wedge-shaped, as shown in Fig. 4, so that they can be driven tightly against the end wood.

Prof. Kidwell recommends that for ordinary purposes an efficiency of 75 per cent. be allowed when oak keys are used and 80 per cent. when the keys are of cast iron. The width of oak keys should be twice the height of the key. Numerous small keys closely spaced gave better results than fewer large keys. In the centre of the span a space equal to about one-quarter of the length of the beam should be left free of keys, bolts, etc. In

TABLE I—SAFE DISTRIBUTED LOADS IN POUNDS FOR COMPOUND KEYED BEAMS.

16 and 20-inch beams to have $1\frac{1}{2} \times 3$-inch oak keys, $\frac{3}{4}$-inch bolts, 3-inch washers.

24-inch beam to have 2×4-inch oak keys, $\frac{7}{8}$-inch bolts, $3\frac{1}{2}$-inch washers.

28-inch beam to have $2\frac{1}{4} \times 4\frac{1}{2}$-inch oak keys, $\frac{7}{8}$-inch bolts, $3\frac{1}{2}$-inch washers.

Size of beam.		Span of beam in feet.					
		20	24	28	30	32	36
1×16	White pine	1,152	960	823	768	720	
	Spruce	1,344	1,120	960	896	840	
	Oregon pine	......	1,440	1,234	1,152	1,080	
	Georgia pine	......	1,600	1,371	1,280	1,200	
1×20	White pine	1,800	1,500	1,285	1,200	1,125	
	Spruce	......	1,750	1,500	1,400	1,312	
	Oregon pine	......	2 250	1,928	1,800	1,687	1,500
	Georgia pine	......	......	2,142	2,000	1,875	1,666
1×24	White pine	......	2,160	1,851	1,728	1,620	1,440
	Spruce	......	2,520	2,160	2,016	1,890	1,680
	Oregon pine	......	......	2,777	2,592	2,430	2,160
	Georgia pine	......	......	3,085	2,880	2,700	2,400
1×28	White pine	......	......	2,520	2,352	2,205	1,960
	Spruce	......	......	......	2,744	2,572	2,286
	Oregon pine	......	......	......	3,528	3,307	2,940
	Georgia pine	......	......	......	3,920	3,675	3,266

To find safe loads for any given thickness of beam, multiply the load in the table by breadth of beam in inches.

For centre loads, take one-half those in table. For concentrated loads at other points divide by the factors given on page 567.

Beams should not be used for shorter or longer spans than those for which safe loads are given, except that 28-inch beams may be used up to 40 feet.

his report, Prof. Kidwell also gives formulas for the number and spacing of the keys.

As compound beams, if used, would probably be built of either 8, 10, 12, or 14-inch timbers, the author has prepared Tables I. and II., giving the maximum safe load that may be allowed for keyed beams 16, 20, 24. and 28 inches in depth, put together as in Figs. 3 and 4, and also the number of keys required on each side of the centre.

TABLE II.—NUMBER OF OAK KEYS REQUIRED EACH SIDE OF CENTRE.

Size of keys.	White pine.	Spruce.	Oregon pine.	Georgia pine.
16-inch beams 1½×3 -inch keys	7	8	11	12
20- " " 1½×3 - " "	9	11	13	15
24- " " 2 ×4 - " "	8	9	12	13
28- " " 2¼×4½- " "	9	10	12	14
Minimum spacing of Keys.				
1½×3 -inch keys	11¼ ins.	11¼ ins.	9 ins.	9 ins.
2 ×4 - " "	15 "	15 "	11½ "	11½ "
2¼×4½- " "	17 "	17 "	13 "	13 "

The breadth or thickness of compound beams should be not less than two-fifths of the depth. The number of keys required is *not affected* by the length or breadth of the beam, if the beam is figured for the full safe load.

In spacing the keys (Fig. 4) they should not be closer than the minimum spacing given in the table. For beams *loaded at the centre*, the spacing of the keys should be *uniform* from X to Y, Y being one-eighth of the span from the centre. If the distance between the keys, centre to centre, works out less than the minimum spacing, the safe load should be correspondingly reduced or the thickness of the beam increased.

For beams uniformly loaded, the first four or five keys from the ends should be spaced for minimum spacing, and the spacing of the remaining keys increased toward the point Y. When the ratio of depth to span is greater than 1 to 16, the inner keys may be a little more than one-eighth of span from centre for distributed loads.

Fig. 3 shows the proper spacing for a 20-inch spruce beam of 28 feet span and for a Georgia pine beam of 30 feet span, and the following table gives the proper spacing for spruce beams (figured from the end of the beam) of longer span. For other woods and spans the spacing should be made as near like these as the fixed

conditions will permit. Four examples of spacing are given below.

The sizes of bolts and washers to be used are given in the heading of Table I. If the beam is not over 10 inches wide the bolts

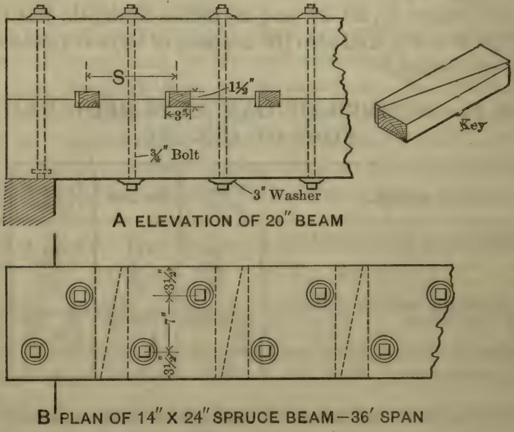


Fig. 4

may be arranged as for the spruce beam, Fig. 3; if 12 inches wide or over the bolts should be staggered as shown for the hard pine beam. In a very wide beam the bolts might be spaced as in detail *B*, Fig. 4.

Spacing of keys in inches (commencing at end) for distributed load:

16-in. spruce beam, 32 feet span,	10, 12,	12,	16,	19, 24, 32.	
20- " " " 32 " "	10, 11½,	11½,	11½,	12, 12, 12, 13, 15, 18, 24.	
24- " " " 36 " "	13, 15,	15,	15,	15, 16, 18, 20, 30.	
28- " " " 36 " "	15, 17,	17,	17,	17, 17, 17, 17, 17, 17,	

Flitch Plate Girders.

A Flitch plate girder is a beam composed of two wooden beams of the same breadth and depth with a wrought-iron or steel plate of the same length and depth as the wooden beams bolted between them, as in Fig. 5. Such beams are much stronger and stiffer than a wooden beam of the same depth, and may often be used in the place of steel beams, where the latter are difficult to obtain.

Flitch plate beams were at one time much used, but with steel at 3 or $3\frac{1}{2}$ cents a pound it is fully as cheap and better to use a steel beam.

The following explanation and formulas are given, however, for the benefit of any one who might have occasion to use a beam of this kind. It has been found by practice that the thickness of the iron plate should be about one-twelfth of the whole thickness of the beam, or the thickness of the wood should be eleven times the thickness of the iron. As the elasticity of iron is so much greater than that of wood, we must proportion the load on the wood so that it shall bend the same amount as the iron plate: otherwise the whole strain might be thrown on the iron plate. The modulus of elasticity of wrought-iron is about thirteen times that of hard pine; or a beam of hard pine one inch wide would bend thirteen times as much as a plate of iron of the same size under the same load. Hence, if we want the hard-pine beam to bend the same as the iron plate, we must put only one-thirteenth as much load on it. If the wooden beam is eleven times as thick as the iron one, we should put eleven-thirteenths of its safe load on it, or, what amounts to the same thing, use a constant only eleven-thirteenths of the strength of the wood. On this basis the following formulas have been made up for the strength of Flitch plate girders, in which the thickness of the iron is one-twelfth of the breadth of the beam, approximately:—

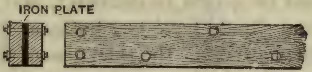


Fig. 5

Let D = Depth of beam.

B = Total thickness of wood.

L = Clear span in feet.

t = Thickness of wrought-iron plate.

$$f = \begin{cases} 82 \text{ pounds for hard pine.} \\ 75 \text{ pounds for Oregon pine.} \\ 60 \text{ pounds for spruce.} \end{cases}$$

W = Total load on girder.

Then, for beams supported at both ends,

$$\text{Safe load at centre, in pounds } = \frac{D^2}{L}(fB + 700t). \qquad (1)$$

$$\text{Safe distributed load, in pounds} = \frac{2D^2}{L}(fB + 700t). \qquad (2)$$

$$\text{For distributed load,} \qquad D = \sqrt{\frac{WL}{2fB + 1{,}400t}}. \qquad (3)$$

$$\text{For load at centre,} \qquad D = \sqrt{\frac{WL}{fB + 700t}}. \qquad (4)$$

The bolts should be ¾-inch in diameter, and spaced 2 feet on centres. Each end should have two bolts, as in Fig. 5.

When steel plates are used, the thickness of the timbers should be about 15 times the thickness of the steel plate. Thus with two 6″×12″ beams the steel plate should be ¾″ thick. Instead of using two beams each 6 ins. thick, three four-inch beams and two ⅜″ plates will generally be better, as it reduces the bending moment on the bolts. If two or three plates are used t should be taken as the total thickness of the plates.

To use the above formulas for steel, multiply t, in formulas 1, 2, and 4 by 900 in place of 700, and in formula 3, by 1,800.

Example.—What is the safe load, uniformly distributed for a girder composed of three 4″×14″ Georgia pine timbers and two ⅜″×14″ Flitch plates, with a span of 25 ft.?

Ans. By formula 2, safe load

$$= \frac{2\times 196}{25}(82\times 12 + 900\times \tfrac{3}{4}) = 26{,}013 \text{ lbs.}$$

TRUSSED BEAMS.

Whenever we wish to support a floor upon girders having a span of more than thirty feet, we must use either a trussed girder, a riveted steel-plate girder, or two or more steel beams. The cheapest and most convenient way is, probably, to use a large wooden girder, and truss it, either as in Figs. 6 and 7, or Figs. 8 and 9.

In all these forms it is desirable to give the girders as much depth as the conditions of the case will permit; as, the deeper the girder, the less strain there is in the pieces.

In the belly-rod truss we either have two beams, and one rod which runs up between them at the ends, or three beams, and two rods running up between the beams in the same way. The beams should be in one continuous length for the whole span of the girder, if they can be obtained that length. The requisite dimensions of the tie-rod, struts, and beam, in any given case, must be determined by first finding the stresses which come upon these

pieces, and then the area of cross-section required to resist these stresses. For SINGLE STRUT BELLY-ROD TRUSSES, such as is represented by Fig. 6, the strain upon the pieces may be obtained by the following formulas:—

For DISTRIBUTED LOAD *W over whole girder,*

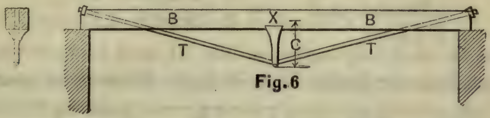


Fig. 6

$$\text{Tension in } T = \tfrac{1}{2}W \times \frac{\text{length of } T}{\text{length of } C}. \quad (5)$$

$$\text{Compression in } C = \tfrac{5}{8}W.* \quad (6)$$

$$\text{Compression in } B = \tfrac{1}{2}W \times \frac{\text{length of } B}{\text{length of } C}. \quad (7)$$

For CONCENTRATED LOAD *W over C,*

$$\text{Tension in } T = \frac{W}{2} \times \frac{\text{length of } T}{\text{length of } C}. \quad (8)$$

$$\text{Compression in } C = W.$$

$$\text{Compression in } B = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } C}. \quad (9)$$

For girder trussed as represented in Fig. 7 *under a* DISTRIBUTED LOAD *W over whole girder,*

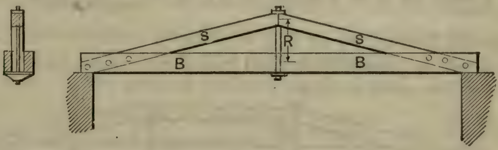


Fig. 7

$$\text{Compression in } S = \tfrac{1}{2}W \times \frac{\text{length of } S}{\text{length of } R} \quad (10)$$

$$\text{Tension in } R = \tfrac{5}{8}W.*$$

$$\text{Tension in } B = \tfrac{1}{2}W \times \frac{\text{length of } B}{\text{length of } R} \quad (11)$$

*When the beam B is in one piece, the full length of span. If B is jointed over the strut then compression in C or tension in $R = \tfrac{1}{2}W$.

For CONCENTRATED LOAD, *W at centre,*

$$\text{Compression in } S = \frac{W}{2} \times \frac{\text{length of } S}{\text{length of } R}. \tag{12}$$

$$\text{Tension in } R = W.$$

$$\text{Tension in } B = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } R} \tag{13}$$

For double strut belly-rod truss (Fig. 8), *with* DISTRIBUTED LOAD *W over whole girder* (beam B divided into three equal spans),

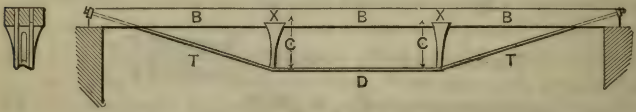


Fig. 8

$$\text{Tension in } T = \frac{W}{3} \times \frac{\text{length of } T}{\text{length of } C}. \tag{14}$$

$$\text{Compression in } C = \frac{W}{3}.$$

$$\text{Comp. in } B \text{ or tension in } D = \frac{W}{3} \times \frac{\text{length of } B}{\text{length of } C}. \tag{15}$$

For CONCENTRATED LOAD *W over each of the struts C,*

$$\text{Tension in } T = W \times \frac{\text{length of } T}{\text{length of } C}. \tag{16}$$

$$\text{Compression in } C = W.$$

$$\text{Comp. in } B \text{ or tension in } D = W \times \frac{\text{length of } B}{\text{length of } C}. \tag{17}$$

For girder trussed, as in Fig. 9, *under a* DISTRIBUTED LOAD *W over whole girder* (beam B divided into three equal spans),

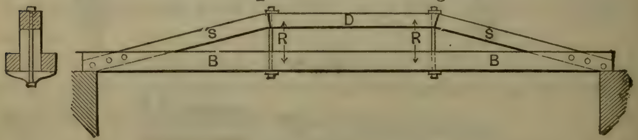


Fig. 9

$$\text{Compression in } S = \frac{W}{3} \times \frac{\text{length of } S}{\text{length of } R}. \tag{18}$$

$$\text{Tension in } R = \frac{W}{3}.$$

$$\text{Tension in } B \text{ or comp. in } D = \frac{W}{3} \times \frac{\text{length of } B}{\text{length of } R}. \tag{19}$$

Under CONCENTRATED LOADS W *applied at* 2 *and* 3.

$$\text{Compression in } S = W \times \frac{\text{length of } S}{\text{length of } R}. \tag{20}$$

$$\text{Tension in } R = W.$$

$$\text{Tension in } B \text{ or comp. in } D = W \times \frac{\text{length of } B}{\text{length of R}}. \tag{21}$$

Trusses as shown in Figs. 8 and 9 should be divided so that the rods R, or the struts C, shall divide the lengths of the girder into three equal or nearly equal parts. The lengths of the pieces T, C, B, R, S, etc., should be measured on the centres of the pieces. Thus the length of R should be taken from the centre of the tie-beam B to the centre of the strut D; and the length of C should be measured from the centre of the rod to the centre of the strut-beam B.

After determining the strains in the pieces by these formulas, we may compute the area of the cross-sections by the following rules:

$$\text{Area of cross-section of short struts} = \frac{\text{comp. in strut}}{C}. \tag{22}$$

The size of the long strut D, Fig. 9, should be determined by means of the tables on pages 411, 412.

The diameter of the tie-rods may be obtained from the table on page 340.

For the beam B, *when the load is distributed,* we must compute its necessary area of cross-section as a tie or strut (according to which truss we use), and also the area of cross-section required to support its load acting as a beam, and give a section to the beam equal to the sum of the two sections thus obtained.

$$\left.\begin{array}{r}\text{Area of cross-section of } B \text{ to} \\ \text{resist tension or compression}\end{array}\right\} = \frac{\text{tension}}{T} \quad \text{or} \quad \frac{\text{comp.}}{C} \tag{23}$$

In trusses 6 and 7, with distributed load,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{W \times L}{4 \times D^2 \times A}. \tag{24}$$

In trusses 8 and 9, with distributed load,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{W \times L}{6 \times D^2 \times A}. \tag{25}$$

W denoting the full distributed load on the girder in pounds, and L the length of one section of the tie-beam in feet. When the

loads are concentrated over C, or at R, then there will be no transverse strain on the beam B, and it need be proportioned only for the tensile or compressive stress, as the case may be.

In formulas 23, 24, and 25,

C = 1,000 pounds per square inch for hard pine and Oregon pine,
800 pounds per square inch for spruce and white oak,
700 pounds per square inch for white pine,
13,000 pounds per square inch for cast-iron.
T = 2,000 pounds per square inch for hard pine and oak,
1,800 pounds per square inch for Oregon pine,
1,600 pounds per square inch for spruce,
1,400 pounds per square inch for white pine,
12,500 pounds per square inch for wrought-iron,
15,000 pounds per square inch for steel.
A = 100 pounds per square inch for hard pine,
90 pounds per square inch for Oregon pine,
70 pounds per square inch for spruce,
60 pounds per square inch for white pine.

Examples.—To illustrate the method of computing the dimensions of the different parts of girders of this kind, we will take two examples.

1. *Computation for a girder such as is shown in Fig.* 6, for a span of 30 feet, the trusses to be 12 feet on centres, and carrying a floor for which we should allow 100 pounds per square foot. The girder will consist of three strut-beams and two rods. We can allow the belly-rod T to come two feet below the beams B, and we will assume that the depth of the beams B will be 12 inches; then the length of C (which is measured from the centre of the beam) would be 30 inches. The length of B would, of course, be 15 feet, and by computation, or by scaling, we find the length of T to be 15 feet $2\frac{1}{2}$ inches.

The total load on the girder equals the span multiplied by the distance of girders on centres, times 100 pounds $= 30 \times 12 \times 100 = 36{,}000$ pounds.

Then, from formula 5,

$$\text{Tension in } T = \frac{36{,}000}{2} \times \frac{182\frac{1}{2} \text{ inches}}{30 \text{ inches}} = 109{,}500 \text{ lbs.}$$

or 54,750 lbs. on each of two rods. For such a large stress it will be best to upset the ends of the rods, and allowing 15,000 lbs.

per square inch for steel rods, we find from the table on page 340 that we must use two $2\frac{1}{4}$-inch steel rods.

The strut-beam we will make of Oregon pine. From formula 7 we find the compressive stress in $B=\frac{36,000}{2}\times\frac{180}{30}=108,000$ pounds. As we are to use three beams, this will give 36,000 lbs. on each beam.

To resist the compression will require $\frac{36,000}{1,000}$ or 36 square ins., which is equal to 3×12 inches.

From formula 24 we find the total breadth required to resist the transverse strain $=\frac{36,000\times15}{4\times144\times90}=12$ ins., or each beam must be 4×12 inches to resist the transverse strain, and 3×12 ins. to resist the compressive strain. Consequently each beam must be 7×12 inches.

As this would make the girder very wide—$25\frac{1}{2}$ ins.—we will use beams 14 ins. deep, increasing the depth of the girder one inch, so that the height on centres will still be 30 ins.

The area required to resist the compressive stress will be the same as before, 36 inches, but as our beam is 14 inches deep the breadth will be only $2\frac{1}{2}$ inches.

The total breadth to resist the transverse strain will be $\frac{36,000\times15}{4\times196\times90}=8$ ins., or $2\frac{2}{3}$ ins. for each beam. The total breadth for each beam will therefore be $5\frac{1}{6}$ inches. A 6×14 beam when dressed will run about $5\frac{1}{2}\times13\frac{3}{4}$ ins., which will just about meet the requirements. The total width of the girder will then be 21 inches. The load on $C=\frac{5}{8}\,W=22,500$ lbs., or 11,250 lbs. over each rod. The sectional area necessary to resist this load $=\frac{11,250}{13,000}$ for cast iron and $\frac{11,250}{800}$ for oak. As the struts must be the full width of the girder, however, it will be necessary to make the sectional area much greater than the theoretical requirements. If made of cast iron the strut should be of the shape shown in Fig. 10, and if of oak, of the shape shown in Fig 11. The cast-iron strut will be the best, but an oak strut will answer.

Example 2.—It is desired to support a floor over a lecture-room forty feet wide, by means of a trussed girder; and as the room above is to be used for electrical purposes it is desired to have a truss with very little iron in it, and hence we use a truss such as is shown in Fig. 9. Where the girders rest on the wall

there will be brick pilasters having a projection of six inches, which will make the span of the truss 39 feet; and we will space the rods *R R* so as to divide the tie-beam into three equal spans of 13 feet each. The tie-beam will consist of two hard-pine beams, with the struts coming between them. We will have two rods, instead of one, at *R*, coming down each side of the strut, and passing through an iron casting below the beams, forming supports for them. The height of truss from centre to centre of timbers we must limit to 18 inches, and we will space the trusses 8 feet on centres. Then the total floor-area supported by one

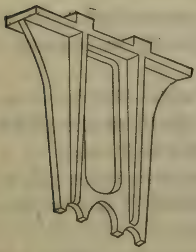


Fig. 10

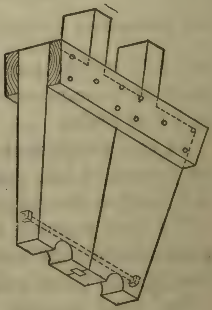


Fig. 11

girder equals 8 feet by 39 feet, equal to 312 square feet. The heaviest load to which the floor will be subjected will be the weight of students, for which 75 pounds per square foot will be ample allowance; and the weight of the floor itself will be about 25 pounds; so that the total weight of the floor and load will be 100 pounds per square foot. This makes the total weight liable to come on one girder 31,200 pounds.

The compression in *S* will be, from formula 18, $\frac{W}{3} \times \frac{157 \text{ ins.}}{18 \text{ ins.}} =$ 90,700 pounds.

Tension in one pair of rods $= \frac{W}{3} = 10{,}400$ pounds.

Tension in *B* or compression in $D = \frac{W}{3} \times \frac{156 \text{ ins.}}{18 \text{ ins.}} = 90{,}130$ lbs.

As the unsupported length of *D* is greater than that of *S*, a beam that will resist the compression in *D* will be ample for *S*. From Table III, page 411, we find that it will require a 10×12

beam 13 feet long to resist the compression in D, a 10×10 not being quite strong enough. The tension in each rod will be only 5,200 lbs., but as the rods must support a large washer at the bottom we will make them 1 in. in diameter, not upset. The tension in each of the beams B will be 45,065 lbs. This divided by $2,000=22.6$ square ins., or say 2×12 ins.

The total breadth of the tie-beam to resist the transverse load we find from formula 25, assuming 12 inches as the depth $=\dfrac{31,065\times13}{6\times144\times100}=4.7$ ins., or about $2\frac{3}{8}$ ins. for each beam.

The breadth of each tie-beam must therefore be $2''+2\frac{3}{8}''=4\frac{3}{8}''$. Hence the tie-beams must be 5×12 ins. Therefore our girder must be built with 10×12 in. strut beams, and two 5×12 in. tie-beams, *each* 42 *ft. long.* The 1-in. rods may be cut $\frac{1}{2}$ in. into the strut-beam, and $\frac{1}{2}$ in. into the tie-beams, so that the latter will come close against the strut S. The "kick" of strut S will be equal to its compressive stress, and we must design a connection between the tie-beams and strut that will be capable of resisting the kick, which in this case is 90,700 lbs. As the inclination of the strut is very slight there will be ample room for bolts. It will be best to use bolts at least $1\frac{1}{2}$ ins. in diameter. As the bolts will be in double shear, the resistance to shearing of one bolt will be (Table V, page 376) 26,500 lbs.

The bearing area of a $1\frac{1}{2}$-inch bolt in a timber 10 inches wide will be 15 inches. For bearing resistance in hard pine we may allow 1,500 lbs. per square inch, which will give 22,500 lbs. as the bearing resistance of one $1\frac{1}{2}''$ bolt. As the force to be resisted is 90,700 lbs. it will require four $1\frac{1}{2}$-inch bolts to sustain the bearing pressure, the resistance to shearing being greater than the stress.

We must now see how many bolts it will require to resist the bending moment. The total bending moment to be resisted (see page 390) $=90,700$ times the distance between the centres of the tie-beams divided by 12, or $90,700\times\dfrac{15}{12}=113,375$ inch-pounds.

From Table IX, Chapter X, we find that the maximum bending moment for a $1\frac{1}{2}$-inch pin is 7,460 lbs. Hence it will require fifteen $1\frac{1}{2}$-inch bolts to resist the thrust in S *without bending the bolts.* It would be impracticable to put in so many bolts, hence we must use larger bolts. For a $2\frac{3}{8}$-inch bolt the maximum bending moment is 29,600 lbs., and four times this gives 118,400 lbs., hence four $2\frac{3}{8}$-inch bolts will be sufficient to resist the bending strain, and also the shearing and bearing stresses. It will be

seen from this example that it is much more difficult and expensive to make satisfactory end joints for girders trussed as in Figs. 7 and 9 than it is for the belly-rod trusses. The belly-rod trusses are to be preferred when the conditions will admit of their use.

These four cases of trussed girders are but special examples of trusses. The stresses in them may also be computed by the methods explained in Chapter XXVI, and where the divisions of the girder cannot be made uniform the stresses should be computed by the general method there explained.

CHAPTER XVIII.

STIFFNESS AND DEFLECTION OF BEAMS.

In Chapters XV. and XVI. we have considered the strength of beams to resist breaking only; but in all first-class buildings it is desired that those beams which show, or which support a ceiling, should not only have sufficient strength to carry the load with safety, but should do so without bending enough to present a bad appearance to the eye, or to crack the ceiling; hence, in calculating the dimensions of such beams, we should not only calculate them with regard to their resistance to breaking, but also to bending. Unfortunately we have at present no method of combining the two calculations in one operation. A beam apportioned by the rules for strength will not bend so as to strain the fibres beyond their elastic limit, but will, in many cases, bend more than a due regard for appearance will justify.

The amount which a beam bends under a given load is called its *deflection*, and its resistance to bending is called its *stiffness;* hence the stiffness is inversely as the deflection.

The rules for the stiffness of beams are derived from those for the deflection of beams; and the latter are derived partly from mathematical reasoning, and partly from experiments.

We can find the *deflection at the centre* of *any* beam not strained beyond the elastic limit, by the following formula:

$$\text{Def. in inches} = \frac{\text{load in lbs.} \times \text{cube of span in inches} \times c}{\text{modulus of elasticity} \times \text{moment of inertia}}. \quad (1)$$

The values of c are as follows:

Beam supported at both ends,		loaded at centre .	0.021
"	"	" uniformly loaded	0.013
"	fixed at one end,	loaded at the other	0.333
"	"	" uniformly loaded	0.125

By making the proper substitutions in Formula 1, we derive

the following formula for a *rectangular* beam *supported at both ends,* and *loaded at the centre*:

$$\text{Def. in inches} = \frac{\text{load} \times \text{cube of span in feet} \times 1{,}728}{4 \times \text{breadth} \times \text{cube of depth} \times E}. \quad (2)$$

From this formula the value of the modulus of elasticity, E, for different materials, has been calculated. Thus beams of known dimensions are supported at each end, and a known weight applied at the centre of the beam. The deflection of the beam is then carefully measured; and, substituting these known quantities in Formula 2, the value of E is easily obtained.

Formula 2 may be simplified somewhat by representing $\frac{1{,}728}{4 \times E}$ by $\frac{1}{F}$, which gives us the formula

$$\text{Def. in inches} = \frac{W \times L^3}{B \times D^3 \times F^*}. \quad (3)$$

For a distributed load the deflection will be five-eighths of this.

If we wish to find the load which shall cause a given deflection, we can transpose Formula 2 so that the load shall form the left-hand member. Thus:

$$\begin{array}{l}\text{Load at centre} \\ \text{in pounds}\end{array} = \frac{4 \times \text{breadth} \times \text{cube of depth} \times \text{def. in ins.} \times E}{\text{cube of span} \times 1{,}728}. \quad (4)$$

Now, that this formula may be of use in determining the load to put upon a beam, the value of the deflection must in some way be fixed. This is generally done by making it a certain proportion of the span.

Thus Tredgold and many other authorities say that if a floor-beam deflects more than one-fortieth of an inch for every foot of span, it is liable to crack the ceiling on the under side; and hence this is the limit which is often given to the deflection of beams in first-class buildings.

Then, if we substitute for "deflection" the value, length in feet $\div 40$, in the above formula, we have,

$$\text{Load at centre} = \frac{\text{breadth} \times \text{cube of depth} \times e}{\text{square of span in feet}}, \quad (5)$$

letting $$e = \frac{E}{17{,}280}.$$

* The constant F corresponds to Hatfield's F, in his Transverse Strains.

Most engineers and architects think that *one-thirtieth of an inch* per foot of span is not too much to allow for the deflection of floor-beams, as a floor is seldom subjected to its full estimated load, and then only for a short time.

If we adopt this ratio, we shall have as our constant for deflection,

$$e_1 = \frac{E}{12,960}.$$

In either of the above cases it is evident that the values used for E, F, e, or e_1, should be derived from tests on timbers of the same size and quality as those to be used. The values for the various woods given in the table below have been adopted by the author after careful comparison with the results of numerous tests on large timbers and with values given by different authorities. The author believes that they are perfectly reliable for first-class merchantable timber.

TABLE I.—VALUES OF CONSTANTS FOR STIFFNESS OR DEFLECTION OF BEAMS.

E = Modulus of elasticity, pounds per square inch.

F = Constant for deflection of beam, supported at both ends, and loaded at the centre.

e = Constant, allowing a deflection of one-fortieth of an inch per foot of span.

e_1 = Constant, allowing a deflection of one-thirtieth of an inch per foot span.

Material.	E.	$F = \frac{E}{432}$	$e = \frac{E}{17,280}$	$e_1 = \frac{E}{12,960}$
Cast iron.	15,700,000	36,300	907	1,210
Wrought iron	26,000,000	60,000	1,500	2,000
Steel.	31,000,000	71,760	1,794	2,358
Long-leaf yellow pine.	1,780,000	4,120	103	137
Oregon pine or Douglas fir	1,425,000	3,300	82	110
Spruce.	1,294,000	3,000	75	100
White pine.	1,073,000	2,480	62	82
Hemlock	1,045,000	2,420	60	80
White oak	1,240,000	2,870	72	95
Ash	1,482,000	3,430	86	114
Chestnut.	944,000	2,180	54	72
Cypress	900,000	2,080	52	69
Maple	1,902,000	4,400	110	146
California red-wood.	700,000	1,620	40	54

Rules for Stiffness of Beams.

Knowing the deflection caused by a weight at the centre of a beam, and the ratio of other deflections, caused by different modes of loading and supporting, we can easily deduce the formulas for the different cases considered under the strength of *rectangular* beams. These cases are:

BEAMS SUPPORTED AT BOTH ENDS.*

Loaded at the centre,

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e}{\text{square of span}}, \tag{6}$$

or,

$$\text{Breadth} = \frac{\text{load} \times \text{square of span}}{\text{cube of depth} \times e}. \tag{7}$$

Loaded at a point other than the centre, m *and* n *being the segments into which the beam is divided,*

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times \text{square of span} \times e}{16 \times m^2 \times n^2}, \tag{8}$$

or,

$$\text{Breadth} = \frac{\text{load} \times m^2 \times n^2 \times 16}{\text{cube of depth} \times \text{square of span} \times e}. \tag{9}$$

Load uniformly distributed,

$$\text{Safe load} = \frac{8 \times \text{breadth} \times \text{cube of depth} \times e}{5 \times \text{square of span}}, \tag{10}$$

or,

$$\text{Breadth} = \frac{5 \times \text{load} \times \text{square of span}}{8 \times \text{cube of depth} \times e}. \tag{11}$$

Inclined beam, loaded at the centre,†

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e}{\text{span} \times \text{hor. dist. between supports}}, \tag{12}$$

---

* In formulas (6) to (17) the breadth and depth are to be taken in inches, and the length or span in feet. The load is always in lbs.

The values given in either of the last two columns of Table I. may be used for *e*, according to the degree of stiffness desired, but the values in the last column are ample under ordinary conditions.

† Tredgold's "Elements of Carpentry," p. 65.

or,

$$\text{Breadth} = \frac{\text{load} \times \text{span} \times \text{hor. dist. between supports}}{\text{cube of depth} \times e}. \quad (13)$$

Beams Fixed at One End.

Loaded at extreme end,

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e}{16 \times \text{square of span}}, \quad (14)$$

or,

$$\text{Breadth} = \frac{16 \times \text{load} \times \text{square of span}}{\text{cube of depth} \times e}. \quad (15)$$

Load uniformly distributed,

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e}{6 \times \text{square of span}}, \quad (16)$$

or,

$$\text{Breadth} = \frac{6 \times \text{load} \times \text{square of span}}{\text{cube of depth} \times e}. \quad (17)$$

Note.—Beams whose span in feet is less than the depth in ins. should *not* be calculated by formulas for stiffness, but by those for strength, Chapter XVI.

Ratio of the Stiffness of Beams.

If the stiffness of a beam supported at both ends and loaded at the centre be called 1
Then that of the same beam with the same load uniformly distributed will be.................................. $\frac{8}{5}$
Firmly fixed at both ends and loaded at the centre, according to Moseley 5
Firmly fixed at both ends and uniformly loaded............ 8
Fixed at one end and loaded at the other .... $\frac{1}{16}$
Fixed at one end and uniformly loaded..................... $\frac{1}{6}$

The stiffest rectangular beam containing a given amount of material is that in which the ratio of depth to breadth is as 10 to 6; hence, in designing beams, the depth and breadth should be made to approach as near this ratio as is practicable.

EXAMPLE 1.—What is the greatest distributed load that an 8 by 10 inch white-pine girder of 12 foot clear span will support, without deflecting at the centre more than $\frac{1}{30}$ of an inch per foot of span?

Ans. This girder comes under the case of a beam supported at both ends and loaded with a uniformly distributed load, and hence should be calculated by Formula 10. Substituting the given dimensions in Formula 10 we have,

$$\text{Safe load} = \frac{8 \times 8 \times 1{,}000 \times 82}{5 \times 144} = 7{,}288 \text{ pounds.}$$

EXAMPLE 2.—What should be the dimensions of a yellow-pine beam of 10-foot span to support a concentrated load of 4,250 pounds, without deflecting more than $\frac{1}{3}$ of an inch at the centre?

Ans. A deflection of $\frac{1}{3}$ of an inch in a span of 10 feet is in the proportion of $\frac{1}{30}$ of an inch per foot of span; and as the load is concentrated and applied at the centre, we should use Formula 7, employing for e the value given in the fourth column opposite yellow pine.

Formula 7 gives the dimensions of the breadth, and to obtain it we must assume a value for the depth. For this we will first try 10 inches.

Substituting in Formula 7 we have,

$$\text{Breadth} = \frac{4{,}250 \times 100}{1{,}000 \times 137} = 3.1 \text{ inches.}$$

Hence it will be necessary to use a $4'' \times 10''$ beam. As the span of this beam in feet is the same as the depth in ins., we should see if it also meets the requirements for strength. From Table III, Chapter XVI, we find that the safe distributed load for a 1×10 beam 10 ft. span is 2,000 lbs., and for a 4×10 beam the safe load would be four times as much, or 8,000 lbs. The load in this example, however, is applied at the centre; hence we must divide the safe distributed load by 2, which gives 4,000 lbs. for the safe centre load. As this is a little less than the load we wish to support, we should increase the size of the beam to 5×10 inches. As a general rule *it is not safe* to use the formulas for stiffness when the span in feet does not exceed the depth in inches.

EXAMPLE 3.—What is the largest load that an inclined spruce beam 8×12 inches, 16 feet long between supports, will carry at the centre, consistent with stiffness, the horizontal distance between the supports being 14 feet?

Ans. Formula 12 is the one to be employed; and we will use the value of e given in the last column opposite spruce. Making the proper substitutions we have,

$$\text{Safe load} = \frac{8 \times 1{,}728 \times 100}{16 \times 14} = 6{,}170 \text{ pounds.}$$

Cylindrical Beams.

For cylindrical beams the same formulas may be employed as for rectangular beams, only instead of e use $1.7 \times e$; that is, a cylindrical beam bends 1.7 times as much as the circumscribing rectangle.

Deflection of Steel Beams.

For rolled steel beams the deflection is most accurately obtained by Formula 1.

A shorter and sufficiently accurate method for determining the deflection of the standard sections under their full safe load is given on pages 510 and 511.

In using steel beams it should be remembered that for any given span the *deepest beam* is always the most economical; and the stiffness of a floor is always greater when a suitable number of deep beams are used.

Tables for Wooden Beams.

The following tables have been prepared so as to show at a glance the greatest load that a beam one inch thick will support without either *exceeding the limit of deflection or the safe strength*. They give the same results as would be obtained by using the above formulas, and the formulas for strength.

To find the corresponding load for thicker beams multiply the load given in the table by the breadth of the beam in inches. The loads thus obtained, however, are for beams that will run *full to the nominal dimensions.* For beams that are ¼-inch scant in

both dimensions the correct load may be obtained by multiplying the load given in the tables by the following factors:

For $1\frac{3}{4}\times 5\frac{3}{4}$	by 1.5		For $1\frac{3}{4}\times 9\frac{3}{4}$	by 1.6
$2\frac{3}{4}\times 5\frac{3}{4}$	" 2.5		$2\frac{3}{4}\times 9\frac{3}{4}$	" 2.55
$1\frac{3}{4}\times 7\frac{3}{4}$	" 1.6		$1\frac{3}{4}\times 11\frac{3}{4}$	" 1.64
$2\frac{3}{4}\times 7\frac{3}{4}$	" 2.5		$2\frac{3}{4}\times 11\frac{3}{4}$	" 2.6
			$1\frac{3}{4}\times 13\frac{3}{4}$	" $1\frac{2}{3}$
			$2\frac{3}{4}\times 13\frac{3}{4}$	" 2.6

Thus the maximum load consistent with stiffness for a $2\frac{3}{4}\times 13\frac{3}{4}$-inch Oregon pine beam of 20 ft. span will be $2.6\times 1{,}207 = 3{,}138$ lbs.

TABLE II.—HARD-PINE BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of Georgia yellow pine *one inch broad*, and supported at both ends, with safety and without deflecting more than one-thirtieth of an inch per foot of span.

For use of table see page 601.

Depth of beam.	Span in feet.											Depth of beam.
	4	6	8	10	12	14	16	18	20	22	24	
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
6	1,800	1,200	738	473	328	242	185	146	118	97	82	6
8	3,200	2,133	1,600	1,121	778	573	438	346	280	231	194	8
9	4,050	2,700	2,025	1,596	1,108	816	624	493	399	329	277	9
10	5,000	3,333	2,500	2,000	1,520	1,120	856	676	548	452	380	10
12	7,200	4,800	3,600	2,880	2,400	1,935	1,479	1,168	950	781	656	12
14	9,800	6,533	4,900	3,920	3,226	2,800	2,348	1,855	1,503	1,240	1,042	14
15	11,266	7,500	5,633	4,500	3,750	3,214	2,816	2,281	1,850	1,525	1,282	15
16	12,800	8,533	6,400	5,120	4,266	3,656	3,200	2,769	2,244	1,851	1,536	16

Loads *above* horizontal lines are calculated by formula for *stiffness*; those *below*, by formula for *strength*.

TABLE III.—OREGON PINE (DOUGLAS FIR) BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of Oregon pine *one inch broad*, and supported at both ends, with safety *and without deflecting more than one-thirtieth of an inch per foot of span.*

For use of table, see page 601.

Depth of beam.	Span in feet.													Depth of beam.
	6	8	10	12	14	15	16	18	20	22	24	25	27	
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
6	1,056	594	380	264	194									
7	1,470	943	603	419	308	268								
8	1,920	1,440	901	626	460	400	352	278	225					8
9	2,430	1,822	1,283	891	654	570	501	396	321	265				9
10	3,000	2,250	1,800	1,222	900	782	687	543	440	361	305			10
12	4,320	3,240	2,592	2,160	1,551	1,351	1,188	938	760	628	528	486		12
14	5,880	4,410	3,528	2,940	2,520	2,146	1,886	1,490	1,207	998	838	772	662	14
15	6,750	5,062	4,050	3,375	2,892	2,700	2,320	1,833	1,485	1,227	1,031	950	814	15
16	7,680	5,760	4,608	3,840	3,291	3,072	2,880	2,225	1,802	1,490	1,251	1,153	990	16

Loads *above* heavy lines are calculated by formula for *stiffness*; those *below* by formula for *strength*.

TABLE IV.—SPRUCE BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of spruce timber *one inch broad*, and supported at both ends, with safety and without deflecting more than one-thirtieth of an inch per foot of span.

For use of table, see page 601.

Depth of beam.	Span in feet.											Depth of beam.
	4	6	8	10	12	14	16	18	20	22	24	
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
6	1,260	540	840	345	240	176	135	106	86	71	60	6
8	2,240	1,493	1,120	819	569	417	320	251	205	170	142	8
9	2,834	1,890	1,417	1,134	810	584	455	359	292	240	202	9
10	3,500	2,333	1,750	1,400	1,111	816	625	493	400	330	277	10
12	5,040	3,360	2,520	2,016	1,680	1,410	1,080	851	691	570	480	12
14	6,860	4,573	3,430	2,744	2,286	1,960	1,715	1,352	1,098	905	762	14
15	7,874	5,250	3,937	3,150	2,625	1,875	1,968	1,663	1,350	1,114	935	15
16	8,960	5,973	4,480	3,584	2,986	2,540	2,240	2,108	1,638	1,352	1,134	16

Loads *above* horizontal lines are calculated by formula for *stiffness*; those *below*, by formula for *strength*.

TABLE V.—WHITE PINE BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of white or common soft pine *one inch broad*, and supported at both ends, with safety *and without deflecting more than one-thirtieth of an inch per foot of span.*

For use of table, see page 601.

Depth of beam.	Span in feet.													Depth of beam.
	6	8	10	12	14	15	16	17	18	19	20	22	24	
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
4	233	131	84											4
6	720	442	283	196										6
7	980	703	450	312	230									7
8	1,280	960	671	466	342	298	262	232						8
10	2,000	1,500	1,200	911	669	583	512	454	405	363	328			10
12	2,880	2,160	1,728	1,440	1,156	1,007	885	784	700	630	567	468	393	12
14	3,920	2,940	2,352	1,960	1,680	1,568	1,406	1,246	1,111	997	900	644	625	14
15	4,500	3,375	2,700	2,250	1,928	1,800	1,687	1,532	1,366	1,226	1,107	914	769	15

Loads *above* heavy lines are calculated by formula for *stiffness*; those *below*, by formula for *strength*

TABLE VI.—OAK BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of white oak *one inch broad*, and supported at both ends, with safety, *and without deflecting more than one-thirtieth of an inch per foot of span.*

For use of table, see page 601.

Depth of beam.	Span in feet.											Depth of beam.
	4	6	8	10	12	14	16	18	20	22	24	
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
6	1,350	900	511	328	226	167	128	101	82	68	57	6
8	2,400	1,600	1,213	778	537	397	303	240	194	160	135	8
9	3,038	2,025	1,519	1,108	765	565	432	342	277	229	192	9
10	3,750	2,500	1,875	1,500	1,050	775	593	470	380	314	264	10
12	5,400	3,600	2,700	2,160	1,800	1,339	1,024	812	656	542	456	12
14	7,350	4,900	3,675	2,940	2,450	2,100	1,627	1,289	1,042	862	724	14
15	8,436	5,625	4,218	3,375	2,812	2,410	2,001	1,586	1,282	1,060	891	15
16	9,600	6,400	4,800	3,840	3,200	2,742	2,560	1,925	1,556	1,286	1,081	16

Loads *above* horizontal lines are calculated by formula for *stiffness*; those *below*, by formula for *strength*.

CHAPTER XIX.

STRENGTH AND STIFFNESS OF CONTINUOUS GIRDERS.

Girders resting upon three or more supports are of quite frequent occurrence in building construction; and a great variety of opinions are held as to the relative strength and stiffness of continuous and non-continuous girders; very few persons, probably, having a correct knowledge of the subject.

In almost every building of importance it is necessary to employ girders resting upon piers or columns placed from eight to fifteen feet apart; and in many cases beams can conveniently be obtained which will span two and even three of the spaces between the piers or columns. When this is the case the question arises, whether it will be better construction to use a long continuous girder, or to have each girder of only one span.

Most architects are probably aware that a girder of two or more spans is stronger and stiffer than a girder of the same section of only one span; but just *how much* stronger and stiffer is a question they are unable to answer.

As it is seldom that a girder of more than three spans is employed in ordinary buildings we shall consider only these two cases. In all structures the first point which should be considered is the resistance required of the supports; and we will first consider the resistance offered by the supports of a continuous girder.

In this chapter we shall not go into the mathematical discussion of the subject, but refer any readers interested in the derivation of the formulas for continuous girders to an article on that subject, by the author, in the July (1881) number of Van Nostrand's "Engineering Magazine."

Supporting Forces.

Girders of Two Equal Spans, Loaded at the Centre of Each Span. —If a girder of two spans, l and l_1, is loaded at the centre of the

span l with W pounds, and at the centre of l_1 with W_1 pounds, the re-action of the support R_1 will be represented by the formula

$$R_1 = \frac{13W - 3W_1}{32}, \tag{1}$$

the re-action of the support R_2 by

$$R_2 = \frac{11}{16}(W + W_1), \tag{2}$$

and the re-action of the support R_3 by the formula

$$R_3 = \frac{13W_1 - 3W}{32}. \tag{3}$$

If $W = W_1$, then each of the end supports would have to sustain $\frac{5}{16}$ of one of the loads, and the centre support $\frac{11}{8}$ of W. Were the girder cut so as to make two girders of one span each, then the end supports would carry $\frac{1}{2}$ or $\frac{8}{16}W$, and the centre support $\frac{16}{16}W$; hence we see that, by having the girder continuous, we do not require so much resistance from the end supports, but more from the central support.

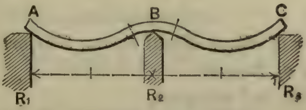


Fig. 1

Girder of Two Spans, Uniformly Distributed Load Over Each Span.—Load over each span equals w pounds per unit of length. Re-action of left support

$$R_1 = \frac{w}{2}\left[l - \frac{l_1^3 + l^3}{4l(l + l_1)}\right]. \tag{4}$$

Re-action of central support,

$$R_2 = w(l + l_1) - R_1 - R_3. \tag{5}$$

Re-action of right support,

$$R_3 = \frac{w}{2}\left[l_1 - \frac{l_1^3 + l^3}{4l_1(l + l_1)}\right]. \tag{6}$$

When both spans are equal to l, the re-action of each end support is $\frac{3}{8}wl$, and of the central support $\frac{5}{4}wl$; hence the girder, by being continuous, reduces the re-action of the end supports, and in-

creases that of the central support by one-fourth, or twenty-five per cent.

Continuous Girder of Three Equal Spans, Concentrated Load of W Pounds at Centre of Each Span.

Re-action of either abutment,

$$R_1 = R_4 = \tfrac{7}{20}W; \tag{7}$$

Re-action of either central support,

$$R_2 = R_3 = \tfrac{23}{20}W; \tag{8}$$

or the re-action of the end supports is lessened three-tenths, and that of the central supports increased three-twentieths of that which they would have been had three separate girders of the same cross-section been used, instead of one continuous girder.

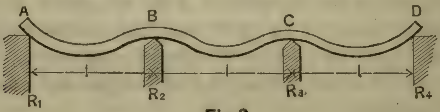


Fig. 2

Continuous Girder of Three Equal Spans Uniformly Loaded with w Pounds per Unit of Length.

Re-action of either end support,

$$R_1 = R_4 = \tfrac{2}{5}wl; \tag{9}$$

Re-action of either central support,

$$R_2 = R_3 = \tfrac{11}{10}wl; \tag{10}$$

hence the re-actions of the end supports are one-fifth less, and of the central supports one-tenth more, than if the girder were not continuous.

Strength of Continuous Girders.—Having determined the re-action of the supports we will now consider the strength of the girder.

The strength of a beam depends upon the material and shape of the beam, and upon the external conditions imposed upon the beam. The latter give rise to the bending-moment of the beam, or the amount by which the external forces (such as the load and supporting forces) tend to bend and break the beam.

It is this bending-moment which causes the difference in the

bearing-strength of continuous and non-continuous girders of the same cross-section.

Continuous Girders of Two Spans.—When a rectangular beam is at the point of breaking we have the following conditions:

$$\text{Bending-moment} = \frac{\text{Mod. of rupture} \times \text{breadth} \times \text{sq. of depth}}{6}; \qquad (11)$$

and, that the beam may carry its load with perfect safety, we must divide the load by a proper factor of safety.

Hence, if we can determine the bending-moment of a beam under any conditions, we can easily determine the required dimensions of the beam from Formula 11.

The greatest bending-moment for a continuous girder of two spans is almost always over the middle support, and is of the opposite kind to that which tends to break an ordinary beam.

Distributed Load.—The greatest bending-moment in a continuous girder of two spans, l and l_1, loaded with a uniformly distributed load of w pounds per unit of length, is

$$\text{Bending-moment} = \frac{wl^3 + wl_1^3}{8(l+l_1)}. \qquad (12)$$

When $l = l_1$, or both spans are equal

$$\text{Bending-moment} = \frac{wl^2}{8}, \qquad (12a)$$

which is the same as the bending-moment of a beam supported at both ends, and uniformly loaded over its whole length; hence a *continuous girder of two spans uniformly loaded is no stronger than if non-continuous.*

Concentrated Load.—The greatest bending-moment in a continuous girder of two equal spans, each of length l, loaded with W pounds at centre of one span, and with W_1 pounds at the centre of the other span, is

$$\text{Bending-moment} = \tfrac{3}{32}l\ (W + W_1). \qquad (13)$$

When $W = W_1$, or the two loads are equal, this becomes

$$\text{Bending-moment} = \tfrac{3}{16}Wl, \qquad (13a)$$

or one-fourth less than what it would be were the beam cut at the middle support.

Continuous Girder of Three Spans, Distributed Load.—The greatest bending-moment in a continuous girder of three spans loaded with a uniformly distributed load of w pounds per unit of length, the length of each end span being l_1 and of the middle

span l, is at either of the central supports, and is represented by the formula,

$$\text{Bending-moment}=\frac{wl^3+wl_1^3}{4(3l+2l_1)}. \tag{14}$$

When the three spans are equal, this becomes

$$\text{Bending-moment}=\frac{wl^2}{10}, \tag{14a}$$

or one-fifth less than what it would be were the beam not continuous.

Concentrated Loads.—The greatest bending-moment in a continuous girder of three equal spans, each of a length l, and each loaded at the centre with W pounds, is

$$\text{Bending-moment}=\tfrac{3}{20}Wl, \tag{15}$$

or two-fifths less than that of a non-continuous girder.

Deflection of Continuous Girders.

Continuous Girder of Two Equal Spans.—The greatest deflection of a continuous girder of two equal spans loaded with a uniformly distributed load of w pounds per unit of length is

$$\text{Deflection}=0.005416\frac{wl^4}{EI}. \tag{16}$$

(E denotes modulus of elasticity; I, moment of inertia.)

The deflection of a similar beam supported at both ends and uniformly loaded is

$$\text{Deflection}=0.013020\frac{wl^4}{EI}.$$

Hence the deflection of the continuous girder is only about two-fifths that of a non-continuous girder. The greatest deflection in a continuous girder is also not at the centre of either span, but between the centre and the abutments.

The greatest deflection of a continuous girder of two equal spans, loaded at the centre of one span with a load of W pounds, and at the centre of the other span with W_1 pounds, is, for the span with load W,

$$\text{Deflection}=\frac{(23W-9W_1)l^3}{1536EI}; \tag{17}$$

for the span with load W_1,

$$\text{Deflection}=\frac{(23W_1-9W)l^3}{1536EI}. \tag{17a}$$

When both spans have the same load,

$$\text{Deflection} = \frac{7}{768}\frac{Wl^3}{EI}. \qquad (17b)$$

The deflection of a beam supported at both ends and loaded at the centre with W pounds is

$$\text{Deflection} = \frac{Wl^3}{48EI},$$

or the deflection of the continuous girder is only seven-sixteenths of the non-continuous one.

Continuous Girder of Three Equal Spans.—Uniformly distributed load of w pounds per unit of length

$$\text{Deflection at centre of middle span} = 0.00052\frac{wl^4}{EI}. \qquad (18)$$

$$\text{Greatest deflection in end spans} = 0.006884\frac{wl^4}{EI}, \qquad (19)$$

or the greatest deflection in the girder is only about one-half that of a non-continuous girder.

Concentrated load of W pounds at centre of each span

$$\text{Deflection at centre of middle span} = \frac{1}{480}\frac{Wl^3}{EI}. \qquad (20)$$

$$\text{Deflection at centre of end spans} = \frac{11}{960}\frac{Wl^3}{EI}; \qquad (21)$$

or only eleven-twentieths of the non-continuous girder.

Several Observations and Formulas for Designing Continuous Girders.

From the foregoing we can draw many observations and conclusions, which will be of great use in deciding whether it will be best in any given case to use a continuous or non-continuous girder.

First as to the Supports.—We see from the formulas given for the re-action of the supporting forces in the different cases that in all cases the end supports do not have as much load brought upon them when the girder is continuous as when it is not; but of course the difference must be made up by the other supports This might often be desirable in buildings where the girders run across the building, the ends resting on the side-walls, and the girders being supported at intermediate points by columns or

piers. In such a case, by using a continuous girder, part of the load could be taken from the walls, and transferred to the columns or piers.

But there is another question to be considered in such a case, and that is vibration. Should the building be a mill or factory in which the girders had to support machines, then any vibration given to the middle span of the beam would be carried to the side-walls if the beam were continuous, while if separate girders were used, with their ends an inch or so apart, but little if any vibration would be carried to the side-walls from the middle span.

In all cases of important construction the supporting forces should be carefully looked after.

Strength.—As the relative strength of continuous and non-continuous girders of the same size and span, and loaded in the same way, is as their bending-moments, we can easily calculate the strength of a continuous girder, knowing the formula for its bending-moment. From the values given for the bending-moments of the various cases considered, we see that the portion of the girder most strained is that which comes over the middle supports; also that, except in the single case of a girder of two spans uniformly loaded, the strength of a girder is greater if it is continuous than if it is not. But the gain in strength in some instances is not very great, although it is generally enough to pay for making the girder continuous.

Stiffness.—The stiffness of a girder is indirectly proportional to its deflection; that is, the less the deflection under a given load the stiffer the girder.

Now, from the values given for the deflection of continuous girders, we see that a girder is rendered very much stiffer by being made continuous; and this may be considered as the principal advantage in the use of such girders.

It is often the case in building construction that it is necessary to use beams of much greater strength than is required to carry the superimposed load, because the deflections would be too great if the beam were made smaller. But, if we can use continuous girders, we may make the beams of just the size required for strength, as the deflections will be lessened by the fact of the girders being continuous. It should therefore be remembered that, where great stiffness is required, continuous beams or girders should be used if possible.

Formulas for Strength and Stiffness.

For convenience we will give the proper formulas for calculating the strength and stiffness of continuous girders of rectangular cross-section. The formulas for strength are deduced from the formula

$$\text{Bending-moment} = \frac{B \times D^2 \times R}{6}, \tag{22}$$

where R is a constant known as the modulus of rupture, and is eighteen times what is generally known as the co-efficient of strength.

STRENGTH.—*Continuous girder of* TWO *equal spans, loaded uniformly over each span,*

$$\text{Breaking-weight}^{*} = \frac{2 \times B \times D^2 \times A}{L}, \tag{23}$$

where B denotes the breadth of the girder, D the depth of the girder (both in inches), and L the length of one span, *in feet.* The values of the constant A are three times the values given in Table II. of Chapter XVI. For yellow pine, 300 pounds; for Oregon pine, 270 pounds; for spruce, 210 pounds; and for white pine, 180 pounds, may be taken as reliable values for A.

Continuous girder of TWO *equal spans, loaded equally at the centre of each span,*

$$\text{Breaking-weight} = \frac{4}{3} \times \frac{B \times D^2 \times A}{L}. \tag{24}$$

Continuous girder of THREE *equal spans, loaded uniformly over each span,*

$$\text{Breaking-weight} = \frac{5}{2} \times \frac{B \times D^2 \times A}{L}. \tag{25}$$

Continuous girder of THREE *equal spans, loaded equally at the centre of each span,*

$$\text{Breaking-weight} = \frac{5}{3} \times \frac{B \times D^2 \times A}{L}. \tag{26}$$

STIFFNESS.—The following formulas give the loads which the beams will support without deflecting more than *one-thirtieth* of an inch per foot of span.

Continuous girder of TWO *equal spans, loaded uniformly over each span,*

$$\text{Load on one span} = \frac{B \times D^3 \times e}{0.26 \times L^2}. \tag{27}$$

---

* Breaking weight in lbs. in all cases.

Continuous girder of TWO *equal spans, loaded equally at centre of each span,*

$$\text{Load on one span} = \frac{16}{7} \times \frac{B \times D^3 \times e}{L^2}. \qquad (28)$$

Continuous girder of THREE *equal spans, loaded uniformly over each span,*

$$\text{Load on one span} = \frac{B \times D^3 \times e}{0.33 \times L^2}. \qquad (29)$$

Continuous girder of THREE *equal spans, loaded equally at the centre of each span,*

$$\text{Load on one span} = \frac{20}{11} \times \frac{B \times D^3 \times e}{L^2}. \qquad (30)$$

The value of the constant e is obtained by dividing the modulus of elasticity by 12,960; and, for the three woods most commonly used as beams, the following values may be taken:

Yellow pine, 137; white pine, 82; spruce, 100; Oregon pine, 110. (For other woods see table, page 597.)

For continuous steel beams the requisite size of beam may be found by first computing the bending-moment, by means of Formulas 12–15, and then selecting a beam whose *section modulus* $= \dfrac{3 \times \text{bending-moment (ft.-lbs.)}}{4{,}000}$. Values for the section modulus for the different shapes of rolled steel used as beams are given in the tables in Chapter X.

EXAMPLE 1.—What size steel beam should be used to support two loads of 16,000 lbs. each, concentrated at the centre of two spans of 10 feet each, the beam being continuous?

Ans. Formula 13*a* gives the bending moment as $\frac{3}{16}Wl$, or 30,000 ft.-lbs. We must therefore use a beam having a section modulus equal to $\dfrac{3 \times 30{,}000}{4{,}000}$ or $22\frac{1}{2}$. From the table on page 297 we find that a 9-inch 30-lb. beam has a section modulus of 22.6, and a 10-inch 25-lb. beam a section modulus of 24. Either of these beams will therefore answer, the 10-inch beam being the cheaper, however.

EXAMPLE 2.—A steel beam continuous over three spans is required to support a distributed load of 1,000 lbs. per linear foot. The two end spans are 12 feet each, and the centre span is 10 feet, what size and weight of I-beam should be used?

Ans. The bending-moment is found by Formula 14, and will be $\frac{1{,}000 \times 1{,}000 + 1{,}000 \times 1{,}728}{4(30+24)} = 12{,}630$.

The section modulus must equal $\frac{3 \times 12{,}630}{4{,}000} = 9.47$, which will require a 7-inch 15-lb. beam.

If the beam were not continuous an 8-inch 18-lb. beam would be required for the 12-foot spans, and a 7-inch beam for the 10-foot span.

For beams of two equal spans, loaded uniformly, the strength of the beam is the same as though the beam were not continuous.

The formulas given for the re-actions of the supports, and for the deflections of continuous girders with concentrated loads, were verified by the author by means of careful experiments on small steel bars. The other formulas have been verified by comparison with other authorities where it was possible to do so; though one or two of the cases given the author has never seen discussed in any work on the subject.

CHAPTER XX.

RIVETED STEEL PLATE AND BOX GIRDERS.

Girders built up of plates and angles, in the manner shown in Figs. 1 to 4, are coming more extensively into use every year. This is undoubtedly owing to the simplicity of their construction, comparatively low cost of the shapes of which they are composed, and their adaptability to any arrangement of loads or to any span for which girders are usually required.

Riveted girders, however, are seldom made of a greater span than 60 feet, nor of a greater height than 5 feet.

The most common forms of these girders are those shown in Figs. 2 and 4.

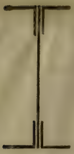


Fig. 1

Fig. 2

Fig. 3

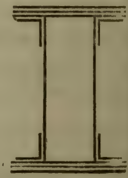


Fig. 4

The sections with a single vertical plate (called the "web") are usually designated as "plate-girders," and those with double or triple webs as "box-girders."

Plate-girders are more economical than box-girders, and more accessible for painting and inspection; but the box-girders are stiffer laterally, and should always be used where great length of span requires a wide top flange.

In general it may be said that plate-girders should be used for supporting floor-beams and floor-arches, and walls not over 12 inches in thickness, and that box-girders should be used where a greater flange width than 12 inches is required.

The section shown in Fig. 1, which has no flange-plates, should only be used for comparatively light loads and short spans, and never for supporting masonry.

The term "flange," as applied to riveted girders, embraces all the metal in top or bottom of girder, exclusive of web-plate.

By the "depth" of a riveted girder is generally meant the distance between the centres of gravity of the flanges; in practice this is taken as the *height of the web-plate*, and the word will be so used in this chapter. The top and bottom of the flange angles are always on a line with the top and bottom of the web-plate.

Stiffeners are short pieces of angles riveted to the web at intervals, to keep the web from buckling. They should fit closely against the horizontal flanges of the flange angles, and should always be used at the supports.

Depth and Width of Girders.—The depth of a riveted girder may be from $\frac{1}{10}$ to $\frac{1}{16}$ of the span. The greatest economy of materials is said to be obtained when the depth is $\frac{1}{12}$ of the span. Thus for a 36-ft. span a 3-ft. girder should be used if the conditions will permit; but the least depth should be $\frac{1}{16}$ of 36, or 2 ft. 3 in.

The width of the top flange should not be less than $\frac{1}{20}$ of the distance between lateral supports; or if there are no lateral supports, then not less than $\frac{1}{20}$ of the span.

Arches between girders or floor beams riveted to the sides of girders may be considered as lateral supports.

Details of Construction.*

1. All the connections and details of the several parts shall be of such strength that, upon testing, rupture shall occur in the body of the members rather than in any of their details or connections.

In members subject to tensile strain full allowance shall be made for the reduction of section by rivet-holes.

2. The webs of plate girders, when they cannot be had in one length, must be spliced at all joints by a plate on each side of the web.

Tees must not be used for splices.

3. Stiffeners shall be used at the ends of all girders and wherever concentrated loads occur, and elsewhere when the shearing strain is greater than the resistance to buckling.

4. The pitch (distance between centres) of rivets shall not exceed 6 in., nor 16 times the thickness of the thinnest outside plate,

* The following twelve points are taken largely from Birkmire's "Compound Riveted Girders."

nor be less than $2\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, or $2\frac{5}{8}$ in. for $\frac{7}{8}$-in. rivets, in a straight line.

5. The rivets used should be $\frac{3}{4}$ in. in diameter for plates from $\frac{3}{8}$ in. to $\frac{5}{8}$ in. thick, and $\frac{7}{8}$ in. in diameter for greater thickness of plates.

6. The distance between the edge of any piece and the centre of a rivet-hole must never be less than $1\frac{1}{4}$ in.

7. In punching plates or other iron, the diameter of the die shall in no case exceed the diameter of the punch more than $\frac{1}{16}$ of an inch.

8. All rivet-holes must be so accurately punched that when the several parts forming one member are assembled together, a rivet $\frac{1}{16}$ inch less in diameter than the hole can be entered, hot, into any hole without reaming or straining the iron by "drifts."

9. The rivets when driven must completely fill the holes.

10. The rivet-heads must be hemispherical, except where flush surfaces are required, and a uniform size for the same-sized rivets throughout the work. They must be full and neatly made, and be concentric to the rivet-holes.

11. Whenever possible, all rivets must be machine-driven.

12. The several pieces forming one built member must fit closely together, and, when riveted, shall be free from twists, bends, or open joints.

Splicing.—"Girders 40 feet and less in length will not require any splicing, as the plates and angles can be readily handled in one length.

"In splicing the top flange, when of two or more thicknesses, no additional cover-plate will be required over the joint, but the ends should be planed true and butt solidly. The rivets to be closer near the joint.

"The plate covering the bottom flange must be of the same area as the plates joined, and of sufficient length to take a number of rivets equal to the strength of the cover-plate."

Calculations for Riveted Girders.

In designing a riveted girder to sustain with safety a given load, the following steps are necessary:

1. To determine the necessary flange area.
2. To determine the thickness of the web to resist (*a*) shearing, (*b*) buckling.

This step also determines whether or not stiffeners are required.

3. To determine the number and pitch of the rivets.

4. To determine the length of the outside flange-plates. When but a single plate is used in the flanges this step is not required.

1. Flange Area.—For determining the flange area of riveted girders, it is customary to assume *that the bending-moment is resisted entirely by the upper and lower flanges,* the web-plate being assumed to resist only the shearing strains. Some engineers include $\frac{1}{6}$ of the section of the web in the flange area, and sometimes the full moment of inertia of the section is taken. The better practice, however, appears to be that based on the assumption first given. The New York Building Law even goes further than this, and requires that "No part of the web shall be estimated as flange area, *nor more than one-half of that portion of the angle-iron which lies against the web.*"

As used in this chapter, the term "flange area" will include the flange or cover-plates, and the full section area (less rivet-holes) of the angles connecting the flange with the web.

In the flange plates and angles subjected to tensile strain full allowance should be made for reduction of section by rivet-holes. For the compression flange the gross sectional area may be taken as making up the same, provided the riveting is well done, so as to completely fill the holes.

The general formula for the strength of beams (see page 500) is: Max. bending-moment = section modulus $\times S$. Assuming that the flanges alone resist the bending-moment the section modulus will be equal to the area of one flange multiplied by the height of the girder and substituting this value in the above equation we have

$$\text{Max. bending-moment} = \text{area of one flange} \times \text{height} \times S,$$

or

$$\left.\begin{array}{c}\text{Area of one flange}\\ \text{in square in.}\end{array}\right\} = \frac{\text{max. bending-moment (ft.-lbs.)}}{\text{height of web in feet} \times S}. \qquad (1)$$

This applies to any condition of loading.

Rules for finding the maximum bending-moment for different conditions of loading are given in Chapter IX.

For the upper or compression flange S should be taken at 12,000 lbs. for steel and 9,000 lbs. for iron.

For the bottom or tension flange S should be taken at 13,000 lbs. for steel and 10,000 lbs. for iron.*

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*Most of the tables giving the strength of riveted girders, found in the recent editions of the manuals issued by the rolling mills, are based on a fibre stress of 15,000 lbs. See pages 648-650.

If it is desired to compute the safe distributed load for a girder already constructed or designed, the following formula may be used:

Safe load in lbs. uniformly distributed =

$$\frac{8 \times \text{net area of bottom flange} \times \text{height in ft.} \times S}{\text{span in feet}}. \qquad (1a)$$

From the result the weight of the girder itself should be subtracted.

For safe centre load take *one-half* the result obtained by formula (1*a*) and subtract weight of girder.

1. Thickness of Web.—The thickness of the web is determined by its resistance to shearing. Whether or not stiffeners shall be used is determined by the resistance of the web to buckling.

SHEARING.—To resist shearing the net sectional area of web must

$$= \frac{\text{maximum shear}}{F}, \qquad (2)$$

F being taken at 6,000 lbs. for iron and 7,000 lbs. for steel.*

The maximum shear in any beam is at one or the other of the supports, and in a girder supported at both ends is equal to *the greater of the supporting forces.*

For a girder supported at both ends and uniformly loaded, maximum shear $= \frac{W}{2}$.

For a girder supported at both ends and loaded at the centre, maximum shear $= \frac{W}{2}$, W representing the total load on the girder.

For a girder supported at both ends and loaded as in Fig. 7, maximum shear $= \frac{W \times m}{L} = P_1$.

For a girder supported at both ends and loaded with two equal concentrated loads W, W, equally distant from the centre, maximum shear $= W$.

For combinations of loads the maximum shear will equal the greater supporting force. The method of determining the supporting forces in a beam is given on pages 274 and 275. The shearing force at *any given vertical section* between the supports

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*These are very conservative values. The Carnegie "Pocket Companion" and several building laws permit 10,000 lbs. for steel.

may be determined by the following rule: *The shearing force at any given cross-section of a beam is the algebraic sum of all the forces acting on the beam from the origin to that cross-section,*

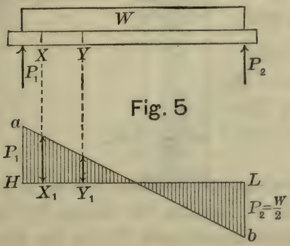

Fig. 5

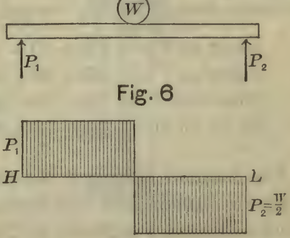

Fig. 6

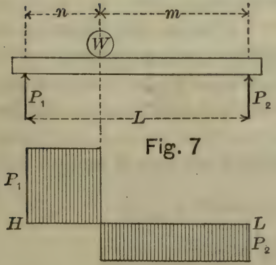

Fig. 7

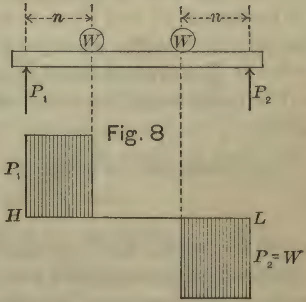

Fig. 8

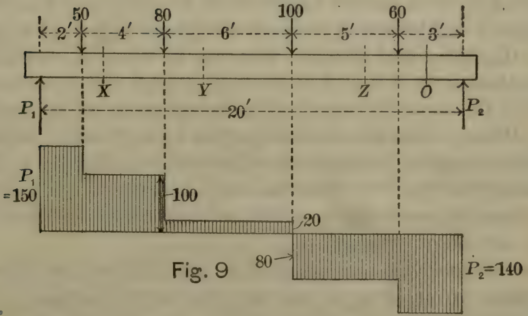

Fig. 9

forces acting upwards being considered as minus, and those acting downwards being considered as plus.

Thus: In the case of the beam shown in Fig. 9 the reaction at P_1 will be found, by the method explained on page 275, to be 150, and that at P_2 to be 140.

Taking our origin at P_1, we would have for the shearing force at the section X, by the foregoing rule,

Shear at $X = -150 + 50 = -100$;
Shear at $Y = -150 + 50 + 80 = -20$;
Shear at $Z = -150 + 50 + 80 + 100 = +80$;
Shear at $0 = -150 + 50 + 80 + 100 + 60 = +140$.

The manner in which the shearing force varies between the supports, under different methods of loading, is shown by the etched areas in Figs. 5–9; in the first three cases W has the same value.

When the load is distributed the shearing force can be found by laying off P_1 and P_2 to a scale of pounds, and drawing the line ab, Fig. 5. The shear at X will then be represented by the ordinate X_1 and the shear at Y by Y_1, which can be readily scaled.

The resistance of the web to buckling is determined by the formula

$$\text{Safe resistance to buckling} = \frac{10{,}000 \times \text{net area of section}}{1 + \dfrac{h^2}{3{,}000t^2}}, \quad (3)$$

where h = height, and t = thickness of web in inches.

When this resistance is less than the shearing force, at any section, stiffeners must be used.

STIFFENERS.—These should be made of angles, not less than $3 \times 3 \times \frac{3}{8}$ in. and seldom larger than $4 \times 4 \times \frac{1}{2}$ in. They should always be tightly fitted between the flange angles, so as to support the horizontal flange. In order to bring the stiffeners in contact with the web and vertical leg of angle, fillers are generally used of the same thickness as the flange angle, as shown in Fig. 10. Where there are several girders exactly alike, something may be saved by omitting the fillers and bending the stiffeners, as shown in Fig. 11. This bending can only be properly done by the use of special dies, and will cost more than the fillers unless there are many stiffeners.

As to the *spacing of stiffeners*, they should in no case (where the resistance to buckling is less than the shear) be spaced farther apart than $1\frac{1}{4}$ times the height of the web, and it is safer to make the distance from centres equal to the height of the web.

On girders supporting distributed loads they are generally placed nearer together at the ends than towards the centre.

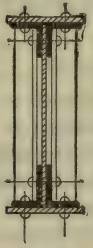

Fig. 10

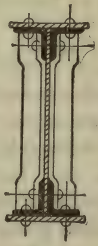

Fig. 11

Stiffeners should *always* be placed at the ends and directly over the edge of the support, as shown in Fig. 18, and wherever concentrated loads occur.

On plate girders the stiffeners are always placed on each side of the web; on box girders generally on the outside only.

Bearing of Girders.—This depends somewhat upon the load, but a safe general rule is to make the bearing of the girder beyond the edge of the support *equal to one-half the height of the girder.*

Spacing of Rivets.

I. Rivets in Web Leg of Angles.

It will readily be seen that when a plate or box girder is loaded, the tendency of the bending-moment is to cause the flange-plates and angles to slide horizontally past the web; this tendency is resisted by the rivets which connect the angles with the web.

The total amount of this tendency to slide (called the "horizontal flange strain"), between any selected point of the flange and the nearer end of the girder, *is equal to the bending-moment at that point divided by the depth of the web.*

The total number of rivets between the selected point and the nearer end must be such that their combined resistance to *shearing* or *bearing* (whichever is the least) shall equal the "horizontal flange strain" at the selected point; or number of rivets

$$= \frac{\text{horizontal flange strain}}{\text{bearing or shearing of one rivet}};$$

and the total number of rivets in web-angle from end to end

$$= \frac{2 \times \text{max. bending-moment (ft.-lbs.)}}{\text{height of web (in ft.)} \times \text{least resistance of one rivet}}. \quad (4)$$

If the number of rivets determined by formula (4) is such that they would be more than 6 in. apart, then the number must be increased, as in no case should they have a greater "pitch" than 6 in.

II. Flange Leg of Angles.

a. *With single cover-plate.*—For girders with a single cover-plate, it is customary to put the same number of rivets in the flange leg as in the web leg for a distance of 3 feet, staggering the rivets as in Fig. 15. Beyond that point to the centre, one-half the number of rivets will be sufficient, provided this will not give them a greater pitch than 6 in.

b. *With two or more cover-plates.*—When two or more cover-plates are used, each plate must have sufficient rivets between the end of the plate and the point where its resistance is required (that is, between a and b, Fig. 13) to transfer to the angle and flange plates between an amount *equal to the safe strength of the plate.* From this point to centre the rivets can be spaced according to the rule for greatest pitch.

III. Rivets in Stiffeners.

The spacing of rivets in the stiffeners is generally determined by the rules given for the pitch of rivets.

Further explanation of the method of determining the spacing of rivets will be found in the following examples.

Approximate Weight of Girder.

In determining the size of a riveted girder to support a given load, it is desirable to be able to add to the superimposed load the weight of the girder itself, as this often forms quite a considerable portion of the load to be supported.

Mr. William H. Birkmire, in his book on "Compound Riveted Girders," gives the following empirical rule for determining the approximate weight of plate or box girders:

$$\text{Weight of girder between support, in tons} = \frac{W \times L}{700}, \tag{5}$$

where W equals load to be supported in tons, and L equals span in feet.

The constant 700 was determined for girders from 35 to 50 ft. long, but may be used without much excess for girders of shorter span.

Tables.

The calculations of riveted girders may be greatly facilitated by Tables I., II., and III. Table I. gives the sectional area that should be deducted for rivet-holes in plates of different thicknesses. In computing this table $\frac{1}{8}$ inch was added to the diameter of the rivet to allow for the injurious effect of punching.

Table II. gives the safe shearing value for web-plates for various depths and thicknesses, and the deduction to be made for each $\frac{3}{4}$-inch or $\frac{7}{8}$-inch rivet.

Table III. gives the safe resistance to buckling per square inch of net section, and also the total resistance of the more common sizes of web-plates, *with two rivet-holes* deducted.

It is very seldom that any vertical section between the stiffeners contains more than two rivet-holes. Tables giving the dimensions of angles will be found in Chapter X., and the shearing and bearing value of rivets is given on page 371.

Examples.

EXAMPLE I.—It is required to support the floor over a room 50×64 feet, by means of riveted steel plate girders, placed across the room and 16 feet on centres. The floor above is to be used for general assembly purposes. The floor joists are of wood, with plastered ceiling below. Design the girder.

First Step: Load.—The first step will be to determine the load to be supported by each girder. The floor area supported by each girder is $50'\times 16'$ or 800 sq. ft. The weight of the floor construction between the girders will be not over 25 lbs. per sq. foot, and an allowance of 100 lbs. per sq. foot for live load will be ample. 800×125 gives 100,000 lbs. or 50 tons as the load to be carried by the girder. To this should be added the weight of the girder itself. The rule for approximate weight of girder is

$$w=\frac{\text{load in tons}\times\text{span}}{700}=\frac{50\times 50}{700}=3.57 \text{ tons, or about } 7{,}000 \text{ lbs.,}$$

making the total load 107,000 lbs. This, of course, will be distributed.

Second Step: Flange Area.—The next step will be to determine the flange area. Before we can do this, however, we must decide upon the width and depth of the girder.

As it is desirable to keep the girder as shallow as possible, consistent with good engineering, we will make the depth, or height, of the web-plate 36 inches, which is about $\frac{1}{16}$th of the span.

As the girders are braced sideways by the floor joists we will make the width of the flange-plates 12 inches.

The flange area is determined by formula (1), and equals

$$\frac{\text{max. bending-moment in ft.-lbs.}}{\text{height of web in ft.} \times S}.$$

The maximum bending-moment for a distributed load equals $\frac{W \times L}{8}$, or in this case

$$\frac{107{,}000 \times 50}{8} = 668{,}750 \text{ ft.-lbs.}$$

Substituting this in the above formula we have,

$$\text{Gross area of upper flange} = \frac{668{,}750}{3 \times 12{,}000} = 18.57 \text{ sq. in.}$$

$$\text{Net area of lower flange} = \frac{668{,}750}{3 \times 13{,}000} = 17.15 \text{ sq. in.}$$

We will first consider the upper flange.

For the angles we will use two $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ angles, with the long leg horizontal.* The area of these two angles we find to be 8 sq. in., which leaves 10.57 sq. in. for the flange-plates. Dividing this by the width of the plate, 12 in., we have .88, or say $\frac{7}{8}$ in. as the required thickness of the plates. We will divide this into two plates, one $\frac{1}{2}$ inch thick, and the upper one $\frac{3}{8}$ inch thick.

We will now see if these plates will have a net area, after deducting the rivet-holes, sufficient for the lower flange. As we shall stagger the rivets in the two legs of the angles, we will have to deduct for only two rivets in flange legs of angles, and two in the flange-plates. We will use $\frac{3}{4}$-inch rivets. From Table I. we find that the area to be deducted for two $\frac{3}{4}$-inch rivets in a $\frac{7}{8}$-inch plate is 1.53, and in a $\frac{1}{2}$-inch plate (thickness of angle) .87. Adding these, we have 2.40 sq. in. to be deducted from 18.50 sq. in., the gross area of upper flange, which leaves 16.1 sq.

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* For the flange-angles of plate girders the $5'' \times 3\frac{1}{2}''$ size is most commonly used, when the flange-plate is from $10\frac{1}{2}$ to 12 inches wide, and $6'' \times 4''$ angles when the flange-plate is over 12 in. wide. For box girders 5×4, $5 \times 3\frac{1}{2}$, $4 \times 3\frac{1}{2}$, and $3\frac{1}{2} \times 3\frac{1}{2}$ are common sizes; while for very heavily loaded girders, requiring two rows of rivets in the web leg, $6'' \times 4''$ angles are often used, with the long leg vertical. For most riveted girders, in which only one row of rivets is required, the short leg is riveted to the web, so as to bring most of the material as far from the centre of the girder as possible. The minimum thickness of flange-angles should be $\frac{3}{8}$ of an inch, and the maximum thickness for ordinary loads $\frac{3}{4}$ inch.

in. as the net area. As this is less than the net area required in the lower flange, we will increase the thickness of outer plate to $\frac{1}{2}$ in., which will give us a little excess of area.

Our flanges will then be made up as follows:

Top flange =	2 angles $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ =	8	sq. in. gross area.
	1 plate $12'' \times \frac{1}{2}''$ =	6	" " " "
	1 plate $12'' \times \frac{3}{8}''$ =	4.5	" " " "
	Total,	18.5	" " " "

Bottom flange =	2 angles $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$; net area	7.13	sq. in.
	2 plates $12'' \times \frac{1}{2}''$ " "	10.25	" "
	Total,	17.38	" "

Third Step: Length of Flange-plates.—We will now determine the length of the outer plate in bottom flange.

To do this we must first determine the horizontal flange strain at centre of girder. This is equal to the bending-moment divided by the height of girder in feet, or $\frac{668,750}{3} = 222,910$ lbs.

The bending-moment in a beam or girder loaded with a uniformly distributed load is represented by the ordinates of a parabola having its height equal to the bending-moment at the centre. The horizontal flange strain is also represented by the same curve.

If, therefore, we draw the parabola rht, Fig. 12 making ah = horizontal flange strain, to a scale of pounds, any ordinate drawn to this curve will represent the flange strain in the girder at the corresponding point on the girder.

To find the theoretical length of the flange-plates, draw the indefinite horizontal line AD, and from the point a lay off the line ad equal to the total flange area, and at such an angle that the end of the line d will come in the line AD. Then divide the line ad into ab = area of two angles; bc = area of first flange-plate; cd = area of second flange-plate.

Draw horizontal lines through b and c; then the line ee' will represent the theoretical required length of the outer flange-plate, and the line ff the length of the first flange-plate.

The plates must in practice, however, be extended beyond the points e and f a distance sufficient to catch enough rivets to transmit at least one-third of the resistance of the plate.

The first flange-plate we will make the full length of the

girder, as it greatly strengthens the angles, and adds but a small amount to the cost of the girder. We will also make the plates in the upper flange of the same length as those in the lower flange.

Fourth Step: Web.—The maximum shearing-stress in a beam uniformly loaded is equal to one-half of the total load, or, in this case, $\frac{107,000}{2}=53,500$ lbs. The web must be thick enough to resist the shear. From Table II. we find that the resistance to shearing of a $\frac{3}{8}'' \times 36''$ web-plate is 94,500 lbs. As this greatly exceeds the total shear, we will use a $\frac{3}{8}$-inch web-plate.

Fifth Step: Stiffeners.—As has been explained, stiffeners will be required wherever the shear exceeds the safe resistance of the

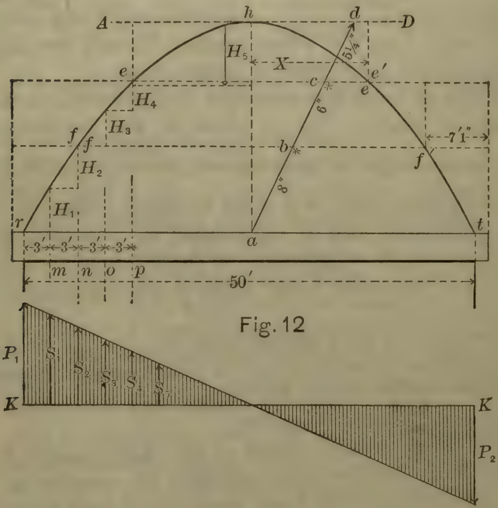


Fig. 12

web to buckling. In this case the maximum shear is 53,500 lbs. and the safe resistance of a $\frac{3}{8}'' \times 36''$ web with two $\frac{3}{4}''$ rivets to buckling (see Table III.) is 31,560 lbs.; hence stiffeners will be required at intervals of about 3 feet.

To determine how many stiffeners will be required, we draw the horizontal line K–K, Fig. 12, and at each end lay off to a scale of pounds the lines P_1, P_2, each equal to one-half the total load on the girder. The shaded triangles will represent the shearing-stress in the web. At 3 ft. from the end the shear will equal S_1; at 6 ft., S_2; at 9 ft., S_3; and so on.

By measuring these lines with our scale we find the shear at m, n, o, p, etc. In this case we find $S_1 = 46{,}200$ lbs., $S_2 = 39{,}900$ lbs., $S_3 = 33{,}600$ lbs., and $S_4 = 27{,}300$ lbs. As S_3 is greater than the safe resistance to buckling, and S_4 less, we might stop the stiffeners at p; but as the floor-joists are framed flush, or nearly so, with the top of the girder, and rest on angles riveted to the web, we will put about 3 stiffeners between the point p and the corresponding point on opposite end.

We will also put a stiffener at each end and directly over each support, so that we will have 15 stiffeners on each side of the girder. These we will make of $4'' \times 4'' \times \frac{3}{8}''$ angles.

Sixth Step: Rivets.—We will first determine the number of rivets in the web leg of the angles. As we should put a rivet in the end of each stiffener, we will determine the number of rivets required between each two adjacent stiffeners. The strain on the rivets between the point m and the end of the girder is equal to the line H_1; between m and $n = H_2$; between n and $o = H_3$; between o and $p = H_4$; and the strain between p and the centre $= H_5$. By scaling these lines we find $H_1 = 48{,}000$ lbs., $H_2 = 44{,}500$ lbs., $H_3 = 38{,}000$, $H_4 = 28{,}500$, and $H_5 = 64{,}410$.

In the web the rivets are in double shear; hence each rivet will have a shearing resistance (see Table II., p. 371) $= 6{,}620$ lbs. The resistance to bearing of a $\frac{3}{4}$ inch rivet on a $\frac{3}{8}$-inch plate (see same table) $= 4{,}220$ lbs., and the latter number will determine the number of rivets.

48,000 lbs. $\div$ 4,220 = 11.4 rivets or 12 rivets, the number required between m and the end of girder. We will put 1 rivet through each of the two end stiffeners, 3 between the end stiffeners, and 7 between the stiffeners at r and m, making 12 rivets to the left of m. Between m and n we must have $\frac{44{,}500}{4{,}220} = 11$ rivets. As this would make them closer together between m and n than between r and m, we will space the 18 rivets evenly between r and n, putting one in the end of the stiffener at n. This will make the pitch just 4 in.

The number of rivets between n and o must equal $\frac{38{,}000}{4{,}220} = 9$, which gives the same pitch as before. Between o and p we must have $\frac{28{,}500}{4{,}220} = 7$ rivets, which gives a pitch of $5\frac{1}{7}$ in.; and between p and the centre we must have $\frac{64{,}410}{4{,}220}$, or 16 rivets. As the dis-

tance is 13 ft., or 156 in., this would make the pitch nearly 10 in. The maximum permissible pitch is 6 in., and we will therefore use that pitch from t to the corresponding point on the other end.

The rivets in the flange leg we will space intermediate with those in the web leg. To determine the length of the outer plate, we have the net area of outer plate in lower flange = 5.13 sq. in. This multiplied by 13,000 = 66,690 lbs., the resistance of the plate. One-third of this, or 22,230 lbs., must be transferred by rivets placed beyond the points e, e', Fig. 12. As the rivets in the flange are in single shear, the shearing strength (3,310 lbs. for $\frac{3}{4}$-inch rivets) will determine the number 22,230 ÷ 3,310 = 7 rivets, or say 3 in each angle.

The point e, Fig. 12, comes at a, Fig. 15, and we will extend the plate so as to take the next three rivets towards the end.

The rivets in the stiffeners we will space as near 6 in. as we can, which gives five rivets between the ends.

The ends of the floor-joist we will support on $4'' \times 4'' \times \frac{1}{2}''$ angles. The load on one lineal foot of this angle, on each side of the girder, = 8 ft. × 125 lbs. = 1,000 lbs.; and as the same rivets support both angles, the total load per running foot will be 2,000 lbs., which is only about one-half of the resistance of a single rivet. We will therefore pitch the rivets about 6 in.

Splices.

As the total length of the girder is 53 ft., it will probably be necessary to splice the web and flange-plates.

The angles should not be spliced, as they can be obtained in one length,* and it is difficult to make a good splice in the angles. If the web is spliced, the joint should be at the centre, as theoretically there is no strain on the web at that point when the load is distributed. We will therefore use for the splice-plates (one on each side of the web) $\frac{1}{2}''$ plates, 8″ wide, and of such length that they will fit closely between the flange angles. These plates will serve as fillers for the middle stiffener. If there was a shearing stress at this point of the web the number of rivets on each side of the joint should be sufficient to transfer the shear from one side of the joint to the other. In this case we will use the same number of rivets as we determined for the stiffener.

The outer flange-plates can easily be obtained in one length. The first flange-plate it may be necessary to splice.

* Angles $3'' \times 3''$ and less can be rolled up to 60 ft. in length. Angles $4'' \times 4''$ to $6'' \times 6''$, up to 50 feet.

Whenever a splice is required in a flange-plate, it should if possible be at a point just beyond the end of the plate above it. The joint must be made by riveting to the spliced plate a plate of the same thickness and of sufficient length to receive a number of rivets on each side of the joint equal to the strength of the plate that is spliced.

When the flange is made up of two plates *of the* same thickness, the simplest method of splicing the inner plate is shown by Fig. 13. Let *e* denote the theoretical end of outer plate, as determined by the strain diagram, and *a* the point to which the plate must be extended to receive rivets equal to one-third the strength of the plate. Then let the joint in inner plate be just under *a* and extend the outer plate to *b*, or such a distance that it can receive a number of rivets equal to the strength of one plate.

In the girder in question we will use a separate splice-plate ½ in. thick and 12 in. wide. The net area of the inner plate is 5.13 sq. in., and its safe strength, at 13,000 lbs. to the inch,

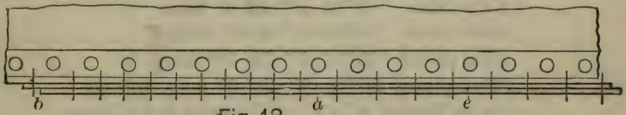


Fig. 13

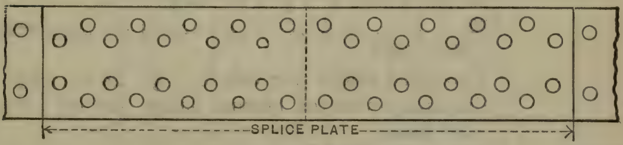


Fig. 14

66,690 lbs., which equals the resistance of 20 rivets. We must therefore have 20 rivets through the splice-plate on each side of

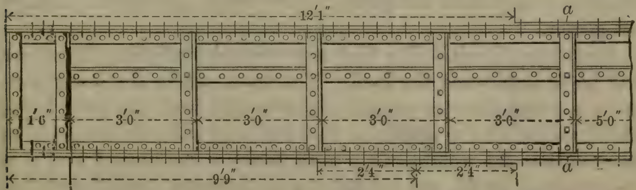


Fig. 15

the joint. These rivets we will space as shown in Fig. 14, which shows the under side of the splice-plate. The splice-plate should

be placed close to the end of the lower plate as shown in Fig. 13. The joint in the upper flange we will make as shown by Fig. 15, extending the outer plate 2′ 4″ beyond *a*.

Fig. 15 shows one end of the girder, drawn according to the foregoing calculations, the joint in the upper flange being at the other end.

For the construction of the girder we shall require the following bill of quantities:

Detail of Girder.

Load 100,000 lbs. uniformly distributed. Span 50′. Depth 3′.

Upper flange.	Two angles	$5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	53 ft. long.	
	One plate	$12'' \times \frac{1}{2}''$	40′ 11″ long.	
	One plate	$12'' \times \frac{1}{2}''$	12′ 1″ long.	
	One plate	$12'' \times \frac{3}{8}''$	30′ 6″ long.	
Lower flange.	Two angles	$5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	53 ft. long	
	One plate	$12'' \times \frac{1}{2}''$	43′ 3″ long.	
	One plate	$12'' \times \frac{1}{2}''$	9′ 9″ long.	
	One plate	$12'' \times \frac{1}{2}''$	28′ 10″ long.	
Web.	Two plates	$36'' \times \frac{3}{8}''$	26′ 6″ long, spliced.	

30 stiffeners $4'' \times 4'' \times \frac{3}{8}''$ angles 2′ 11″ long.
28 filler-plates $4'' \times \frac{1}{2}'' \times 29''$ long.
92 ft. 8 in. of $4'' \times 4'' \times \frac{1}{2}''$ angles for supporting floor-joist.
Two splice-plates for web $8'' \times \frac{3}{8}''$ 29 in. long.
One splice-plate on bottom flange $12'' \times \frac{1}{2}''$, 4 ft. 8″ long.

Rivets ...... $\frac{3}{4}$ in. in diameter.

Example II.—It is required to support the wall shown in Fig. 16 by a riveted steel box girder at the height indicated. Design the girder.

First Step: Loads.—The first step towards designing the girder will be to determine the load. The space under the lower windows is too shallow to permit of the weight from the piers being distributed over the girder, so that the only safe calculation is to assume that the weight of the wall between the lines *A* and *B* is concentrated at W_1, the weight of wall between lines *B* and *C* at W_2, and so on.

We will assume that the floor-joists run across the building so that only the weight of the wall will be supported by the girder.

Allowing 200 lbs. as the weight of one square foot of a 21-inch wall plastered on the inside, and 165 lbs. per sq. ft. for the 17-inch wall, we shall have:

Load at W_1

$$= \left\{ \begin{array}{ll} [5'\ 3'' \times 10' - 7' \times 2'\ 3''] \times 200 = \ldots\ldots\ldots\ldots & 7{,}350 \\ [5'\ 3'' \times 40' - (2'\ 3'' \times 14' + 3'\ 2'' \times 7')] \times 165 = & 25{,}795 \end{array} \right\} \begin{array}{l} 33{,}145 \\ \text{lbs.} \end{array}$$

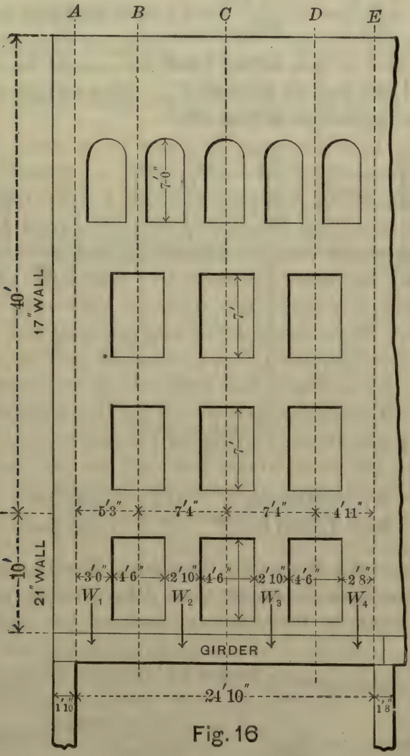


Fig. 16

Load at W_2

$$= \left\{ \begin{array}{ll} [7'\ 4'' \times 10' - 4'\ 6 \times 7'\ 0''] \times 200 = \ldots\ldots\ldots\ldots & 8{,}366 \\ [7'\ 4'' \times 40' - (4'6 \times 14' + 4'9'' \times 7')] \times 165 = & 32{,}354 \end{array} \right\} \begin{array}{l} 40{,}720 \\ \text{lbs.} \end{array}$$

Load at W_3 = that at W_2 = ...40,720 lbs.

Load at W_4

$$= \left\{ \begin{array}{l} [4'\ 11''\times 10' - 2'\ 3''\times 7']\times 200 = \ldots\ldots\ldots 6{,}683 \\ [4'\ 11''\times 40' - (2'\ 3''\times 14' + 3'\ 2''\times 7')] = 23{,}595 \end{array} \right\} 30{,}278 \text{ lbs.}$$

Total load on girder= .... 144,863 lbs.
or 72.4 tons

Approximate weight of girder

$$= \frac{W\times L}{700} = \frac{72.4\times 24\frac{5}{6}}{700} = 2.5 \text{ tons or } 5{,}000 \text{ lbs.}$$

About one-third of this, or say 1,600 lbs., should be added to W_2 and W_3, and 900 lbs. to W_1 and W_4. This will give us the following loads applied as in Fig. 17:

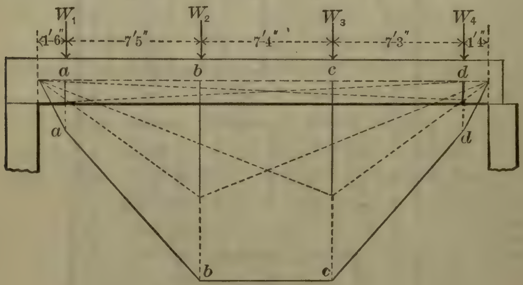


Fig. 17

$W_1 = 34{,}000$ lbs.; $W_2 = 42{,}300$ lbs.;
$W_3 = 42{,}300$ lbs.; $W_4 = 31{,}200$ lbs.

Second Step: To Determine Maximum Bending-moment.—By means of the formula under Case VI., p. 269, we find the bending-moment (in foot-lbs.) for each of the loads to be as follows:

$$\text{Bending-moment for } W_1 = \frac{34{,}000\times 1'\ 6''\times 23'\ 4''}{24'\ 10''} = 47{,}980 \text{ ft.-lbs.}$$

$$\text{Bending-moment for } W_2 = \frac{42{,}300\times 8'11''\times 15'11''}{24'\ 10''} = 242{,}000 \text{ ft.-lbs}$$

$$\text{Bending-moment for } W_3 = \frac{42{,}300\times 16'\ 3''\times 8'\ 7''}{24'\ 10''} = 237{,}900 \text{ ft.-lbs.}$$

$$\text{Bending-moment for } W_4 = \frac{31{,}200\times 1'\ 4''\times 23'\ 6''}{24'\ 10''} = 39{,}420 \text{ ft.-lbs.}$$

Platting these moments to a scale, as explained on page 271, we

get the diagram shown in Fig. 17.* The maximum bending-moment is at W_2, and its amount equals the line bb, which scales 418,000 ft.-lbs.

Third Step: To Determine Flange Area and Length of Cover-plates.—Before we can determine the flange area we must decide upon the height of the web-plate. As we have plenty of room for our girder, we will make the height of the web-plate 30 in., or about $\frac{1}{10}$th of the span.

Then by formula (1),

$$\text{Gross area of upper flange} = \frac{418{,}000}{2.5 \times 12{,}000} \text{ or very nearly 14 sq. in.;}$$

$$\text{Net area of lower flange} = \frac{418{,}000}{2.5 \times 13{,}000} \text{ or 13 sq. in.}$$

As the thickness of the wall to be supported is 21 inches, we will use flange-plates 20 in. wide. The least thickness of cover-plate that we should use is $\frac{3}{8}$ in. The area of a $\frac{3}{8} \times 20$-inch plate is $7\frac{1}{2}$ sq. in., which leaves $6\frac{1}{2}$ sq. in. to be made up by the angles. The area of two $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$ angles is 7.06, which gives a little excess for the top flange. In the lower flange we must allow for two $\frac{3}{4}$-inch rivet-holes in both the plate and angles. From Table I., we find that the area to be deducted for two $\frac{3}{4}''$ rivets, in a $\frac{3}{8}''$ plate, is 0.65, and in a $\frac{7}{16}$-inch plate 0.76 or 1.41 sq. in. in all. The gross area of the plate and angles is $7\frac{1}{2} + 7.06 = 14.56$ sq. in. Deducting 1.41 sq. in., we have 13.15 sq. in. for net area of bottom flange, which is just above what we require.

We will therefore use the same size of plate and angles in both flanges. As the width of the flange is more than one-twentieth of the span, the girder will not require lateral support.

Fourth Step: Web and Stiffeners.—The least thickness that should be used for the web-plates is $\frac{3}{8}$ in. We will therefore first determine if this thickness is sufficient to resist the shearing stress.

The maximum shearing stress will be at the left-hand support, and will equal the reaction of that support. This reaction we must determine by formula (2), page 275, or

$$P_1 = \left\{ \frac{\begin{array}{l} 34{,}000 \times 23'\ 4'' + 42{,}300 \times 15'\ 11'' \\ \qquad + 42{,}300 \times 8'\ 7'' + 31{,}200 \times 1'\ 4'' \end{array}}{24'\ 10''} \right\} = 75{,}450 \text{ lbs.}$$

As there are two web-plates, the shearing force to be resisted by each plate will be one-half of this, or 37,725 lbs. From Table

* The bending-moments in this diagram are drawn to the scale of 400,000 ft.-lbs. to the inch.

II. we find that the resistance of a steel plate $\frac{3}{8} \times 30$ in. to shearing is 78,750 lbs., or twice as great as the stress. From Table III. we find the safe resistance to buckling, deducting for two $\frac{3}{4}$-inch rivets, is 33,830 lbs. As this is a little less than the shear, we will place 4×4 stiffeners, 2′ 4″ from each support, and 5 between them, about 3′ 4″ on centres.

[NOTE.—If our loads were really concentrated at the points W_1, W_2, etc., as by the action of column or girder, it would be necessary to put stiffeners at each of those points, and two in each of the 7-foot spaces. But as in this case the load is partially distributed it will be better to space them as above indicated.]

There will also be two stiffeners over each support.

The only remaining point to determine is the

SPACING OF RIVETS.

We will first determine the number of rivets in the web leg of angle, between W_1 and left-hand end. The bending-moment at W_1 is found by scaling the line *aa*, which gives 110,000 lbs. This divided by the height of the girder gives 44,000 lbs., which is the horizontal flange strain at that point. As there are two webs, only one-half of this strain, or 22,000 lbs., will come on one angle. The rivets in this case will be in single shear. From table on page 371, we find that the resistance of a $\frac{3}{4}$ rivet to single shear is 3,310 lbs., and the bearing value on a $\frac{3}{8}''$ plate 4,220 lbs. The resistance to shearing will therefore determine the number of rivets. $22,000 \div 3,310$ gives 7 as the number of rivets required in the web leg of each angle between W_1 and the left-hand end of girder. This distance is 40 in., which would make the pitch about $5\frac{1}{2}$ in.

Over the bearings, however, the pitch ought not to exceed 4 in.; we will therefore increase the number of rivets to 12, spacing the first six $3\frac{3}{8}$ in. on centres, and the next seven 4 in. on centres.

We will next determine the number of rivets between W_1 and W_2. The horizontal flange strain at $W_2 = \frac{418,000}{2.5} = 167,200$ lbs., and one-half of this is 83,600 lbs. Dividing 83,600 by 3,310, we have 26 as the number of rivets required between W_2 and the left-hand end of girder. As we have already put in 12 rivets we shall only need $26 - 12$ or 14 rivets between W_1 and W_2 to resist the horizontal strain. This distance is 89 in., which divided by 14 gives $6\frac{5}{14}$ in. for the pitch. As for practical reasons

the pitch should not exceed 6 in., we will give the rivets that pitch from W_1 to the corresponding distance from the other end, making both ends of the girder alike.

Theoretically the number of rivets between W_2 and the right-hand end should be the same as on the other side of W_2, and most of these would be required at the right of W_3, but in this case the number of rivets is determined by the maximum pitch.

The number of rivets theoretically required in the flange leg of angle equals the safe strength of plate divided by 3,310. The safe strength of a $\frac{3}{8} \times 20$ inch plate at 13,000 lbs. unit stress $=$ $13{,}000 \times \frac{3}{8} \times 20 = 97{,}500$ lbs. This divided by 3,310 gives 30 rivets for both angles. As this number is so small, we must be guided by the rule of maximum pitch, and will use the same number of rivets, less one, as in the web leg, staggering the rivets.

The girder will then be detailed as below:

DETAIL OF GIRDER.

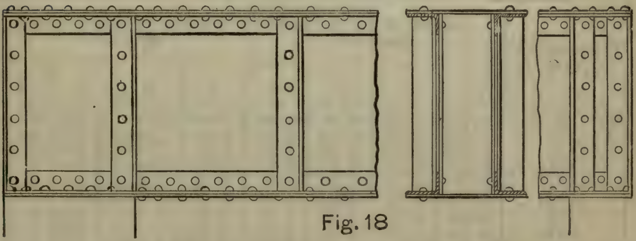


Fig. 18

Loads 34,000 lbs. 1′ 6″ from left support. Span 24′ 10″.
" 42,300 " 8′ 11″ from left support. Depth 30″.
" 42,300 " 8′ 7″ from right support.
" 31,200 " 1′ 4″ from right support.
Both flanges: 4 angles, $5' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, 27′ 6″ long.
One plate, $20'' \times \frac{3}{8}''$, 27′ 6″ long.
Two webs, $\frac{3}{8}'' \times 30''$, 27′ 6″ long.
22 stiffeners, $4'' \times 4'' \times \frac{3}{8}''$, $29\frac{1}{8}''$ long.
22 filler-plates, $4'' \times \frac{7}{16}''$, 23″ long.
Rivets, $\frac{3}{4}$ inch diameter.

By the rules and examples above given it is possible to compute the necessary dimensions and details for riveted girders under any conditions of loading. If further examples are desired, the reader is referred to "Compound Riveted Girders," by William H. Birkmire, in which eight different examples of loading are fully worked and explained.

Detail drawings, with strain diagrams of one of the heaviest plate girders ever used in building construction, are given in the *Engineering Record* of Dec. 28, 1895. This girder is one of six plate girders used in the construction of the Tremont Temple, Boston, Mass., Messrs. Blackall & Newton, architects. The girder is 75 ft. long between centres of columns 6 ft. 1 in. high, with flanges 28 in. wide, and was calculated to support distributed and concentrated loads aggregating 497.5 tons. The single web-plate is $64\frac{3}{4}$ in. high, and $\frac{7}{8}$ in. thick at the ends; the flanges are $4\frac{1}{2}$ in. thick at centre; flange angles, $6'' \times 8'' \times 1''$.

TABLE I.—SECTIONAL AREA TO BE DEDUCTED FROM PLATES AND ANGLES FOR RIVET-HOLES.

(BIRKMIRE.)

Taken ⅛ inch in excess of diameter of rivet.

Thickness of plate.	Number of rivets, 1 inch diameter.				Number of rivets, ⅞ inch diameter.			
	1	2	3	4	1	2	3	4
1	1.12	2.25	3.37	4.50	1.00	2.00	3.00	4.00
15/16	1.05	2.10	3.16	4.21	0.94	1.87	2.81	3.75
7/8	0.98	1.97	2.95	3.93	0.87	1.75	2.62	3.50
13/16	0.91	1.83	2.74	3.65	0.81	1.62	2.44	3.25
3/4	0.84	1.69	2.53	3.37	0.75	1.50	2.25	3.00
11/16	0.77	1.55	2.32	3.09	0.69	1.37	2.06	2.75
5/8	0.70	1.41	2.11	2.81	0.62	1.25	1.87	2.50
9/16	0.63	1.26	1.90	2.53	0.56	1.12	1.69	2.25
1/2	0.56	1.11	1.69	2.25	0.50	1.00	1.50	2.00
7/16	0.49	0.98	1.47	1.97	0.44	0.87	1.31	1.75
3/8	0.42	0.84	1.26	1.69	0.37	0.75	1.12	1.50

Thickness of plate.	Number of rivets, ¾ inch diameter.				Number of rivets, ⅝ inch diameter.			
	1	2	3	4	1	2	3	4
1	0.87	1.75	2.62	3.50	0.75	1.50	2.25	3.00
15/16	0.82	1.64	2.46	3.28	0.70	1.40	2.11	2.81
7/8	0.77	1.53	2.30	3.06	0.65	1.31	1.96	2.62
13/16	0.71	1.42	2.13	2.84	0.61	1.22	1.83	2.44
3/4	0.66	1.31	1.96	2.62	0.56	1.12	1.69	2.25
11/16	0.60	1.20	1.80	2.40	0.51	1.03	1.54	2.06
5/8	0.55	1.09	1.64	2.19	0.47	0.94	1.41	1.88
9/16	0.49	0.98	1.48	1.96	0.42	0.84	1.26	1.69
1/2	0.43	0.87	1.31	1.75	0.37	0.75	1.12	1.50
7/16	0.38	0.76	1.15	1.53	0.33	0.66	0.98	1.31
3/8	0.32	0.65	0.98	1.31	0.28	0.56	0.84	1.12
5/16	0.27	0.55	0.82	1.09	0.23	0.47	0.70	0.94
1/4	0.22	0.44	0.66	0.87	0.18	0.37	0.56	0.75

TABLE II.—SHEARING VALUE OF WEB-PLATES IN POUNDS.

Wrought Steel. Gross Area. Unit Stress 7,000 lbs.

Depth in inches.	Thickness in inches.						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	73,500	85,750	98,000	110,250	122,500	147,000	171,500
30	78,750	91,875	105,000	118,125	131,250	157,500	183,750
32	84,000	98,000	112,000	126,000	140,000	168,000	196,000
36	94,500	110,250	126,000	141,750	157,500	189,000	220,500
40	105,000	122,500	140,000	157,500	175,000	210,000	245,000
42	110,250	128,625	147,000	165,375	183,750	220,500	257,250
46	120,750	140,875	161,000	181,125	201,250	241,500	281,750
48	126,000	147,000	168,000	189,000	210,000	252,000	294,000
	Deduct for one ¾-inch rivet.						
	2,240	2,660	3,010	3,430	3,850	4,620	5,390
	Deduct for one ⅞-inch rivet.						
	2,625	3,080	3,500	3,920	4,375	5,250	6,125

EXAMPLE.—What is the safe shearing value of a $36'' \times \frac{3}{8}''$ web-plate with seven $\frac{3}{4}$-inch rivets in stiffeners?

Answer.—Gross shearing value $= 94,500$ lbs.

Deduct for 7 rivets $7 \times 2,240 = 15,680$ "

Safe resistance $= 78,820$ lbs.

TABLE III.—SAFE BUCKLING VALUE OF WEB-PLATES IN POUNDS PER SQUARE INCH.

Calculated by formula $p = \dfrac{10000}{1 + \dfrac{d^2}{3000t^2}}$.

d = depth in inches. t = thickness in inches.

Depth in inches.	Thickness in inches.						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	3,498	4,228	4,890	5,476	5,932		
30	3,192	3,896	4,546	5,133	5,656	6,522	
32	2,889	3,624	4,228	4,787	5,339	6,226	6,920
36	2,456	3,069	3,666	4,229	4,748	5,656	6,392
40	2,087	2,696	3,191	3,724	4,228	5,133	5,882
42	1,930	2,455	2,983	3,498	3,992	4,889	5,649
48	1,548	1,994	2,543	2,918	3,371	4,228	4,992

TOTAL RESISTANCE FOR PLATES WITH TWO $\frac{3}{4}''$ RIVETS.

Depth in inches.	Thickness in inches.						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	34,450	48,580	64,200	80,880	97,340		
30	33,830	48,150	64,230	81,560	99,880	138,200	
36	31,560	46,000	62,800	81,500	101,750	145,300	191,570
42	29,140	43,230	60,040	79,190	100,440	147,600	198,960
48	26,860	40,360	58,820	75,920	97,450	146,670	202,000

TOTAL RESISTANCE FOR PLATES WITH TWO $\frac{7}{8}''$ RIVETS.

Depth in inches.	Thickness in inches.						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	34,100	48,110	63,570	80,100	96,390		
30	33,510	47,720	63,640	80,840	98,980	136,960	
36	31,310	45,660	62,320	80,900	100,690	144,230	190,170
42	28,950	42,960	59,660	78,700	99,800	146,690	197,710
48	26,700	40,140	58,490	75,520	96,910	145,860	200,930

Tables of Riveted Steel Plate and Box Girders.

The tables on pages 644–647, giving the greatest safe distributed load that should be imposed on the girders were computed by the author in accordance with formula ($1a$), using 13,000 lbs. fibre strain and deducting for two $\frac{7}{8}$-inch holes (for $\frac{3}{4}$-inch rivets) in flange-plates and angles.

From the safe load given by the formula *the weight of the girder between supports has been subtracted.*

The sizes given are those most commonly found in buildings.

The weight per foot should be considered as a close approximation only; it is intended to include rivets and stiffeners, where required.

These tables should be used only for determining the size of girder and thickness of the plates and angles. Rivet spacing should be determined in each case by the rules previously given, and also the number and position of stiffeners. For distributed loads below the heavy cross-lines stiffeners will not be required. It should, however, be remembered that stiffeners are always required at the ends of the girders, as shown in Figs. 15 and 18.

These tables may also be used to serve as a check on special calculations for other girders. *If more than two rivet-holes* occur in any given cross-section of the bottom flange, then the safe load must be decreased accordingly.

The loads given in the second column of the tables are for the *least thickness* of flange-plates that should be used. This thickness may be *increased* as required to give the desired strength to the girder, thus: If we wish to carry a distributed load of 47 tons, with a span of 35 feet, with a girder of the dimensions given for girder A, we must increase the thickness of the flange-plates sufficient to take difference between 47 and 38.12 tons or 8.88 tons. This will require an increase of $\frac{4}{16}$ or $\frac{1}{4}$ inch in both top and bottom flange.

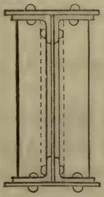
It is not desirable to use girders of these dimensions for greater spans than those given in the table

The tables on pages 648–650 are from the manual of the Passaic Rolling Mill Co., prepared by Geo. H. Blakeley, C.E. It should be noted that they are based on a fibre stress of 15,000 lbs. per sq. inch, and that they include weight of girders.

STEEL PLATE GIRDERS.

SAFE LOADS IN TONS UNIFORMLY DISTRIBUTED.

See explanation, page 643.



A

30″ × ½″ web-plate.
12″ × ⅜″ flange-plates.
5″ × 3½″ × ½″ angles.
Weight per foot, 154 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
20	69.80	4.16
21	66.33	3.96
22	63.17	3.78
23	60.27	3.61
24	57.82	3.46
25	55.16	3.33
26	52.90	3.20
27	50.79	3.08
28	48.83	2.97
29	47.00	2.87
30	45.28	2.77
31	43.68	2.68
32	42.17	2.60
33	40.74	2.52
34	39.40	2.44
35	38.12	2.38
36	36.92	2.31
37	35.77	2.25
38	34.69	2.19
39	33.66	2.13
40	32.66	2.08

Max. load for ½″ web, 86.94 tons.

B

33″ × ½″ web-plate.
12″ × ⅜″ flange-plates.
5″ × 3½″ × ½″ angles.
Weight per foot, 162 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
20	76.81	4.57
21	73.00	4.36
22	69.52	4.16
23	66.34	3.98
24	63.42	3.81
25	60.72	3.66
26	58.22	3.52
27	55.91	3.39
28	53.75	3.27
29	51.74	3.15
30	49.86	3.05
31	48.09	2.95
32	46.43	2.86
33	44.86	2.77
34	43.38	2.69
35	41.98	2.61
36	40.66	2.54
37	39.12	2.46
38	38.20	2.41
39	37.06	2.35
40	35.97	2 29

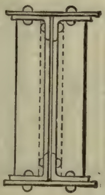
Max. load for ½″ web, 94.43 tons.

Loads above heavy cross-line require stiffeners.

STEEL PLATE GIRDERS.

SAFE LOADS IN TONS UNIFORMLY DISTRIBUTED.

See explanation, page 643.

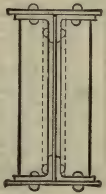


C

36″ × ½″ web-plate.
12″ × ⅜″ flange-plates.
5″ × 3½″ × ½″ angles.
Weight per foot, 168 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
20	83.88	4.99
24	69.28	4.16
26	63.63	3.84
28	58.76	3.56
30	54.52	3.33
31	52.60	3.22
32	50.78	3.12
33	49.08	3.02
34	47.48	2.93
35	45.95	2.85
36	44.51	2.77
37	43.14	2.70
38	41.84	2.63
39	40.59	2.56
40	39.42	2.49
41	38.29	2.43
42	37.21	2 37
44	35.19	2.27
46	33.34	2.17
48	31.83	2.08
50	30.02	1.99

Max. load for ½″ web, 104 tons.
Max. load for 9/16″ web, 117 tons.



D

42″ × ½″ web-plate.
12″ × ⅜″ flange-plates.
5″ × 3½″ × ½″ angles.
Weight per foot, 180 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
24	81.02	4.85
26	74.45	4.48
28	68.78	4.16
30	63.85	3.88
31	61.61	3.75
32	59.51	3.64
33	57.53	3.53
34	55.66	3.42
35	53.89	3.32
36	52.21	3.23
37	50.63	3.15
38	49.11	3.06
39	47.68	2.98
40	46.31	2.91
41	45.00	2.84
42	43.75	2.77
44	41.41	2.64
46	39.26	2.53
48	37.27	2.42
50	35.43	2.33
55	31.35	2.11

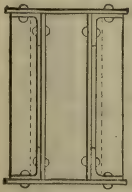
Max. load for ½″ web, 122 tons.
Max. load for ⅝″ web, 152 tons

Loads above heavy cross-line require stiffeners.

STEEL BOX GIRDERS.

Safe Loads in Tons Uniformly Distributed.

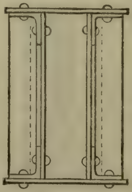
See explanation, page 643.



E 30″ × 3/8″ web-plates.
16″ × 3/8″ flange-plates.
3½″ × 3½″ × ½″ angles.
Weight per foot, 170 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
20	69.60	5.78
21	66.13	5.51
22	62.95	5.26
23	60.04	5.03
24	57.58	4.82
25	54.91	4.63
26	52.64	4.45
27	50.52	4.28
28	48.55	4.13
29	46.70	3.99
30	44.98	3.85
31	43.36	3.73
32	41.84	3.61
33	40.40	3.50
34	39.05	3.40
35	37.76	3.30
36	36.55	3.21
37	35.38	3.12
38	34.30	3.04
39	33.25	2.96
40	32.25	2.89

Max. load for 3/8″ webs, 130 tons.



F 36″ × 3/8″ web-plates.
16″ × 3/8″ flange-plates.
3½″ × 3½″ × ½″ angles.
Weight per foot, 200 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
20	83.56	6.94
24	68.90	5.78
26	63.21	5.34
28	58.31	4.96
30	54.04	4.62
31	52.10	4.48
32	50.27	4.34
33	48.55	4.20
34	46.93	4.08
35	45.39	3.96
36	43.93	3.85
37	42.55	3.75
38	41.23	3.65
39	39.97	3.56
40	38.78	3.47
41	37.63	3.38
42	36.54	3.30
44	34.49	3.15
46	32.60	3.02
48	31.06	2.89
50	29.22	2.77

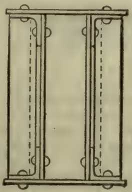
Max. load for 3/8″ webs, 157 tons.

Loads above heavy cross-line require stiffeners.

STEEL BOX GIRDERS.

Safe Loads in Tons Uniformly Distributed.

See explanation, page 643.

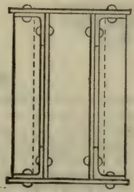


G

30″ × 3/8″ web-plates.
20″ × 1/2″ flange-plates.
3½″ × 3½″ × ½″ angles.
Weight per foot, 212 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
20	93.75	7.41
21	89.08	7.06
22	84.83	6.73
23	80.93	6.44
24	77.35	6.17
25	74.05	5.95
26	70.99	5.70
27	68.16	5.49
28	65.51	5.29
29	63.05	5.11
30	60.73	4.94
31	58.56	4.78
32	56.53	4.63
33	54.60	4.49
34	52.80	4.36
35	51.07	4.23
36	49.44	4.11
37	47.90	4.00
38	46.43	3.90
39	45.03	3.80
40	43.69	3.70

Max. load for 3/8″ webs, 130 tons.



H

36″ × 3/8″ web-plates.
20″ × 1/2″ flange-plates.
4″ × 4″ × 1/2″ angles.
Weight per foot, 236 lbs.

Span in feet.	Safe applied load, in tons.	Increase in safe load for 1/16″ increase in thickness of flange-plates.
20	120.49	8.89
24	99.54	7.41
26	91.43	6.84
28	84.45	6.35
30	78.36	5.92
31	75.60	5.73
32	73.00	5.55
33	70.56	5.39
34	68.25	5.23
35	66.07	5.08
36	64.00	4.94
37	62.03	4.80
38	60.18	4.68
39	58.40	4.56
40	56.70	4.44
41	55.08	4.34
42	53.54	4.23
44	50.65	4.04
46	47.98	3.86
48	45.52	3.70
50	43.24	3.55

Max. load for 3/8″ webs, 157 tons.

Loads above heavy cross-line require stiffeners.

STEEL PLATE GIRDERS.

PASSAIC ROLLING MILL CO.

SAFE LOADS IN TONS OF 2,000 LBS. UNIFORMLY DISTRIBUTED.

No stiffeners required except at ends, over supports only.

Girders equivalent to a 24″ I beam.

Web........ Angles......	24″×⅜″ 5″×3½″×½″		26″×⅜″ 5″×3½″×7/16″		28″×⅜″ 5″×3½″×⅜″		30″×⅜″ 5″×3″×⅜″	
Span, centres of bearings, feet.	Safe load, tons.	Increase for 1/16″ increase in thickness of angles.	Safe load, tons.	Increase for 1/16″ increase in thickness of angles.	Safe load, tons.	Increase for 1/16″ increase in thickness of angles.	Safe load, tons.	Increase for 1/16″ increase in thickness of angles.
20	47.2	5.3	46.5	5.8	45.1	6.2	47.7	6.4
21	44.9	5.0	44.3	5.5	42.9	5.9	45.5	6.1
22	42.9	4.8	42.3	5.2	41.0	5.7	43.4	5.8
23	41.0	4.6	40.4	5.0	39.2	5.4	41.5	5.5
24	39.3	4.4	38.8	4.8	37.6	5.2	39.8	5.3
25	37.7	4.2	37.2	4.6	36.1	5.0	38.2	5.1
26	36.3	4.1	35.8	4.4	34.7	4.8	36.7	4.9
27	34.9	3.9	34.4	4.3	33.4	4.6	35.4	4.7
28	33.7	3.8	33.2	4.1	32.2	4.5	34.1	4.5
29	32.5	3.6	32.1	4.0	31.1	4.3	32.9	4.4
30	31.4	3.5	31.0	3.8	30.0	4.2	31.8	4.2
31	30.4	3.4	30.0	3.7	29.1	4.0	30.8	4.1
32	29.4	3.3	29.1	3.6	28.2	3.9	29.8	4.0
33	28.6	3.2	28.2	3.5	27.3	3.8	28.9	3.9
34	27.7	3.1	27.4	3.4	26.5	3.7	28.1	3.7
35	26.9	3.0	26.6	3.3	25.8	3.6	27.3	3.6
36	26.2	2.9	25.8	3.2	25.0	3.5	26.5	3.5
37	25.5	2.8	25.1	3.1	24.4	3.4	25.8	3.4
38	24.8	2.8	24.5	3.0	23.7	3.3	25.1	3.3
39	24.2	2.7	23.8	2.9	23.1	3.2	24.5	3.3
40	23.6	2.6	23.3	2.9	22.5	3.1	23.9	3.2
Weight per foot, lbs.	88	7.2	84	7.2	79	7.2	79	6.8

Safe loads given include weight of girder.

Weights of girders given include weight of rivet heads, but not stiffeners.

Maximum fibre strain, 15,000 lbs. per square inch of net area, holes for ¾″ rivets being deducted.

STEEL PLATE GIRDERS.

PASSAIC ROLLING MILL CO.

Safe Loads in Tons of 2,000 Lbs. Uniformly Distributed.

No stiffeners required except at ends, over supports only.

Girders equivalent to two 24″ I beams.

Web........ Angles...... Plates.......	24″ × 9/16″ 5″ × 5″ × 1/2″ 12″ × 1/2″		26″ × 9/16″ 5″ × 5″ × 7/16″ 12″ × 1/2″		28″ × 1/2″ 5″ × 5″ × 3/8″ 12″ × 1/2″		30″ × 1/2″ 5″ × 5″ × 3/8″ 12″ × 3/8″	
Span, centres of bearings, feet.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.
20	90.8	3.6	93.6	3.9	93.6	4.3	91.7	4.6
21	86.5	3.4	89.1	3.7	89.1	4.1	87.3	4.3
22	82.5	3.3	85.1	3.6	85.0	3.9	83.4	4.1
23	78.9	3.1	81.3	3.4	81.3	3.7	79.7	3.9
24	75.6	3.0	78.0	3.3	78.0	3.6	76.4	3.8
25	72.6	2.9	74.8	3.1	74.8	3.4	73.3	3.6
26	69.8	2.8	72.0	3.0	72.0	3.3	70.5	3.5
27	67.2	2.7	69.3	2.9	69.3	3.2	67.9	3.4
28	64.8	2.6	66.8	2.8	66.8	3.1	65.5	3.3
29	62.6	2.5	64.5	2.7	64.5	3.0	63.2	3.1
30	60.5	2.4	62.4	2.6	62.4	2.9	61.1	3.0
31	58.6	2.3	60.4	2.5	60.4	2.8	59.2	2.9
32	56.7	2.2	58.5	2.5	58.5	2.7	57.3	2.8
33	55.0	2.2	56.7	2.4	56.7	2.6	55.6	2.8
34	53.4	2.1	55.0	2.3	55.0	2.5	53.9	2.7
35	51.9	2.0	53.5	2.3	53.5	2.4	52.4	2.6
36	50.4	2.0	52.0	2.2	52.0	2.4	50.9	2.5
37	49.1	1.9	50.6	2.1	50.6	2.3	49.6	2.5
38	47.8	1.9	49.2	2.1	49.2	2.3	48.3	2.4
39	46.6	1.8	48.0	2.0	48.0	2.2	47.0	2.3
40	45.4	1.8	46.8	2.0	46.8	2.1	45.8	2.3
Weight per foot, lbs.	158	5.1	153	5.1	143	5.1	136	5.1

Safe loads given include weight of girder.

Weights of girders given include weight of rivet heads, but not stiffeners.

Maximum fibre strain, 15,000 lbs. per square inch of net area, holes for 3/4″ rivets being deducted.

STEEL BOX GIRDERS.

PASSAIC ROLLING MILL CO.

Safe Loads in Tons of 2,000 Lbs. Uniformly Distributed.

No stiffeners required except at ends, over supports only.

Girders equivalent to two 24″ **I** beams.

Webs. ...... Angles. ..... Plates.	24″ × 3/8″ 5″ × 3″ × 1/2″ 14″ × 9/16″		26″ × 3/8″ 5″ × 3″ × 7/16″ 14″ × 1/2″		28″ × 3/8″ 5″ × 3″ × 3/8″ 14″ × 7/16″		30″ × 3/8″ 5″ × 3″ × 3/8″ 14″ × 3/8″	
Span, centres of bearings, feet.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.	Safe load, tons.	Increase for 1/16″ increase in thickness of plates.
20	93.8	4.3	93.5	4.7	92.9	5.1	95.6	5.4
21	89.3	4.1	89.0	4.5	88.5	4.8	91.1	5.2
22	85.3	3.9	85.0	4.3	84.5	4.6	86.9	4.9
23	81.6	3.8	81.3	4.1	80.8	4.4	83.2	4.7
24	78.2	3.6	77.9	3.9	77.4	4.2	79.7	4.5
25	75.0	3.5	74.8	3.8	74.3	4.1	76.5	4.3
26	72.2	–3.3	71.9	3.6	71.5	3.9	73.6	4.2
27	69.5	3.2	69.2	3.5	68.8	3.8	70.8	4.0
28	67.1	3.1	66.8	3.4	66.3	3.6	68.3	3.9
29	64.7	3.0	64.4	3.2	64.0	3.5	66.0	3.7
30	62.5	2.9	62.3	3.1	61.9	3.4	63.8	3.6
31	60.5	2.8	60.3	3.0	60.0	3.3	61.7	3.5
32	58.6	2.7	58.4	2.9	58.1	3.2	59.8	3.4
33	56.9	2.6	56.6	2.8	56.3	3.1	58.0	3.3
34	55.2	2.5	55.0	2.7	54.6	3.0	56.3	3.2
35	53.6	2.5	53.4	2.7	53.1	2.9	54.7	3.1
36	52.1	2.4	51.9	2.6	51.6	2.8	53.1	3.0
37	50.7	2.3	50.5	2.5	50.2	2.7	51.7	2.9
38	49.4	2.3	49.2	2.5	48.9	2.7	50.3	2.9
39	48.1	2.2	47.9	2.4	47.6	2.6	49.0	2.8
40	46.9	2.2	46.7	2.4	46.4	2.6	48.0	2.8
Weight per foot, lbs.	174	6.0	166	6.0	159	6.0	158	6.0

Safe loads given include weight of girder.

Weights of girders given include weight of rivet heads, but not stiffeners.

Maximum fibre strain, 15,000 lbs. per square inch of net area, holes for 3/4″ rivets being deducted

CHAPTER XXI.

STRENGTH AND STIFFNESS OF WOODEN FLOORS.

Two problems present themselves under this head; first, to proportion the beams and girders forming the framework of the floor to the greatest load likely to come upon it; and second, to determine the maximum safe load for a floor already built.

The former of these problems is the one with which architects and builders more commonly have to deal, and will therefore be considered first.

Layout of the Floor Framing.—Before any calculations can be made for the size of the timbers it will be necessary to know the span of the joists, and, if there are openings in the floor, or the floor-joists have to support longitudinal partitions, a framing plan should be made, showing the floor area that will be supported by each beam, and also the position of partitions or special loads. If the floor is to be supported by posts and girders the position of these should also be accurately indicated on the framing plan. For a detailed description of the manner of framing wooden floors the reader is referred to Part II. of "Building Construction and Superintendence."

Where the floor-beams are supported entirely by walls or partitions, the span of the beams will of course be fixed by the plan of the building. When the distance between walls and partitions is too great for a single span, there may be a question as to the best location of the posts and girders.

When planning a building in which wooden floor-beams are to be used, it is important to keep in mind how the floors are to be framed, and particularly the span. Whenever practicable the span of wooden beams should be kept under 25 feet. When the distance between the supporting walls exceeds 30 feet, girders should be placed so that the maximum span of the joists will not exceed 24 feet for light buildings or 16 to 18 feet for warehouses.

In school buildings it is desirable to have the rooms at least 27

feet wide, and hence in this class of buildings the joists usually have a span of from 27 to 30 feet. For a span of 30 feet, however, 16-inch joists should be used, and as these are expensive, and often difficult to obtain, it is much better and more economical to make the schoolrooms 27×32 or 34 feet, than to make them 30 feet square. In the opinion of the writer a schoolroom 27 feet wide by 32 to 34 feet long, with windows on the long side of the room only, is the most economical and satisfactory, as it permits of using 3″×14″ joists 28 feet long, and also gives the most satisfactory lighting.

When floor-beams are supported by a girder placed so that a 24- or 26-foot beam will reach over the two spans, it is always better to have the joists continuous over the girder, as it makes a much stiffer floor, although the ultimate strength is not increased (see Chapter XIX.).

Having decided on the arrangement of the joists, and drawn a framing plan showing the span and location of all special timbers, the next step will be to decide upon the loads for which the joist and timbers shall be proportioned.

Floor loads are made up of two factors, first the weight of materials composing the floor (and ceiling below, if there is one); and second, an allowance for the load liable to come upon the floor. The first is commonly designated as the "dead load," and the second as the "live load." When the "safe load" for a floor is spoken of the live load is generally meant.

Weight of Wooden Floor Construction.—Wooden floors usually consist of beams, commonly called "joists," or "floor-joists," one or two thicknesses of flooring boards, and, in a finished building, of a ceiling underneath the beams. In figuring the weight of $\frac{7}{8}$-inch flooring boards it will be sufficiently accurate to estimate the weight of a single thickness at 3 pounds per square foot. The joists may also be figured at 3 pounds per foot, board measure, with the exception of hard pine and oak joists, which should be figured at 4 pounds per foot board measure. The weight of the joists must also be reduced to their equivalent weight per square foot of floor. Thus the weight of a 2×12-inch joist is about 6 pounds per lineal foot. If the joists are spaced 12 inches on centres, this will be equal to 6 pounds per square foot; but if the joists are 16 inches on centres there will be but one lineal foot of joist to every $1\frac{1}{3}$ square feet, which will be equivalent to $4\frac{7}{8}$ pounds per square foot, and if they are 20 inches on centres, the weight will be equal to $3\frac{1}{8}$ pounds per square foot;

spaced 24 inches on centres, the weight will be 3 pounds per square foot.

The weight of a lath-and-plaster ceiling should be taken at 10 pounds per square foot, and of a $\frac{3}{4}$-inch wood ceiling at $2\frac{1}{2}$ pounds per square foot. Corrugated iron ceiling weighs about 1 pound per square foot. For stamped steel ceilings 2 pounds per square foot will cover the weight of the metal and furring.

The following table, giving the weight of joists, will be found convenient in figuring the weight of floors:

TABLE I.—WEIGHT OF FLOOR-JOISTS PER SQUARE FOOT OF FLOOR.

Size of joists in inches.	Spruce, hemlock, white pine.		Hard pine or oak.	
	Spacing in inches, centre to centre.		Spacing in inches, centre to centre.	
	12	16	12	16
	Pounds.	Pounds.	Pounds.	Pounds.
2×6....	3	$2\frac{1}{4}$	4	3
2×8.....	4	3	$5\frac{1}{3}$	4
3×8.....	6	$4\frac{1}{2}$	8	6
2×10....	5	$3\frac{3}{4}$	$6\frac{2}{3}$	5
3×10....	$7\frac{1}{2}$	$5\frac{1}{2}$	10	$7\frac{1}{2}$
2×12....	6	$4\frac{1}{2}$	8	6
3×12....	9	$6\frac{3}{4}$	12	9
2×14....	7	$5\frac{1}{4}$	$9\frac{1}{3}$	7
3×14....	$10\frac{1}{2}$	$8\frac{1}{2}$	14	$10\frac{1}{2}$

Weight of Crowds.—Prof. L. J. Johnson, of Harvard Univ., reports in the *Eng. News* of Apr. 14, 1904, results of some tests to ascertain the weight of crowds (of men), in which he obtained weights of 134.2, 143.9, 148.1, and 156.9 lbs. per sq. ft. The last weight was obtained by packing 67 men in a room about 11′×6′. Prof. Johnson also found that with 50 men in the room, giving a weight of 122 lbs. per sq. ft., the crowd was compacted "so that a man could elbow his way through it only with perseverance and determined effort."

Superimposed Loads.—There is much difference of opinion as to what allowance should be made for the live load. Table II. shows the minimum allowance for live loads for different classes of buildings, as fixed by the building laws of the cities mentioned:

TABLE II.—MINIMUM SAFE SUPERIMPOSED LOADS FOR FLOORS REQUIRED BY VARIOUS BUILDING LAWS.

Class of buildings.	Minimum live load per square foot of floor.					
	Buffalo, 1896.	Boston, 1895.	Chicago, 1895.	Denver, 1898.	New York, 1899.	St. Louis, 1897.
Dwellings	40	50	70	40	60	70
Hotels, tenements, and lodging-houses	70	50	70	50†	60	70
Offices	70	100	70	70	75*	70*
Buildings for public assembly	100	150	70	80‡	90	120†
Stores, warehouses, and mfg. bldgs.	120	250	150§	150§	120§	150§

* First floor, 150 lbs.
† Also schoolhouses.
‡ With fixed desks.
§ And upwards.

It is the opinion of the author that the following allowances for floor loads, taken in connection with the values given for the safe strength of beams, will provide absolute safety with proper allowance for economy.

For dwellings, sleeping and lodging rooms	40 lbs.
For schoolrooms	50 "
For offices (upper stories)	60 "
For offices (first story)	80 "
For stables and carriage-houses	65 "
For banking-rooms, churches, and theatres	80 "
For assembly halls, dancing halls, and the corridors of all public buildings, including hotels	120 "
For drill-rooms	150 "

Floors for ordinary stores, light manufacturing and light storage should be computed for not less than 120 pounds per square foot, and to sustain a concentrated load at any point of 4,000 lbs.

It is rarely, if ever, that the floors of a dwelling, tenement, or lodging-house, or the rooms in a hotel, are loaded to more than twenty pounds per square foot, for the entire area, and a minimum load of 40 pounds should provide for all possible contingencies.

The floors of offices are as a rule not more heavily loaded than dwellings, but the possibilities for increased loads, in the way of safes and heavy furniture, and possibly of a more compact crowd of people, are greater, so that the minimum floor load for offices should be somewhat increased. Some years ago Messrs. Blackall

& Everett, of Boston, found that the average live load in 210 offices, in three prominent office buildings in that city, was between 16 and 17 pounds, while the average load for the 10 heaviest offices was 33.3 pounds. As such loads, however, are not usually evenly distributed, some portions of the floor being generally much more heavily loaded than the others, it would not appear to be safe to use the average above determined for determining the strength of floor-beams and arches, although it would probably answer for the columns. There seems to be considerable difference of opinion among the leading architects and structural engineers as to just what allowance should be made for office floors. In the Mills Building in San Francisco the live loads were assumed at 40 pounds per square foot for all floors above the first; in the Venetian Building, Chicago, the second, third, and fourth floors were calculated for 60 pounds, and the upper floors for 35 pounds live load per square foot, while in the Old Colony and Fort Dearborn Buildings in Chicago the live loads on the floor-beams were assumed at 70 pounds in accordance with the building ordinance.

An allowance of 120 lbs. per square foot for the live load in churches, theatres, and schoolhouses is, in the opinion of the author, much greater than the actual conditions require.

The average size of a schoolroom is about 28×32 feet, and such a room usually contains seats for fifty-six scholars and the teacher. Assuming the average weight of each scholar at 120 pounds, we have for the average live load, including ten visiting adults and the desks and furniture, 13 pounds per square foot. Even supposing that the scholars of two rooms were united for some special occasion, we would have but 21 pounds per square foot, and this is as great a load as it is possible to imagine in such a room, as the fixed desks prevent the crowding together of the scholars except at the sides of the room. From this reasoning, therefore, a minimum load for the schoolrooms of 50 pounds per square foot would appear abundant.

As a matter of fact, 3×14-inch Georgia pine beams, 16 inches on centres and 28 feet span, have been used for schoolroom floors for years, and no practical person would doubt their safety, but such beams, if calculated by the formula for stiffness as hereinafter recommended, would only support a live load of 56 pounds.

The minimum floor space allotted to a single seat in theatres is 4 square feet, while the average is about 5 square feet. As-

suming the weight of an opera-chair at 35 pounds and of the average adult at 140 pounds (a liberal allowance), we have an average of 44 pounds per square foot of floor. A minimum of 80 pounds would therefore seem to provide for any possible crowding during a panic except in corridors. On the other hand it has been shown * that a crowd of able-bodied men may produce a load of about 120 pounds per square foot, and this should be the minimum for assembly halls, without fixed desks, and also for the corridors of all public buildings. For armories the minimum load should be increased on account of the vibration produced.

The average floor loads for stores has also been greatly overestimated. Mr. W. L. B. Jenney found that the average load on the floors of the wholesale warehouse of Marshall Field & Co., in Chicago, was but 50 pounds per square foot, and very few retail stores will average over 80 pounds. An allowance of 120 pounds is sufficient for any ordinary retail store, with the possible exception of hardware stores.

Warehouses, on the other hand, may be very heavily loaded, and the floors in buildings intended for the storage of merchandise should be proportioned to the especial class of goods which they are designed to support.

The following table, originally compiled by Mr. C. J. H. Woodbury,† and to which some additions have been made by the Insurance Engineering Experiment Station and by the author, will be found of assistance in deciding upon the live load to be assumed for warehouse floors. The weights per square foot are for single packages. If the goods are piled two or more cases high, the weight per square foot of floor will of course be increased accordingly. In fact, the height to which the goods are liable to be piled is a very important consideration in fixing upon the floor load.

In the following table "the measurements were always taken to the outside of case or package, and gross weights of such packages are given."

To find the size of joists, beams, and girders required for any particular building.—As already explained, the first step should be to make a framing plan of the floors or enough of it to show any special framing and the span

* See "Weight of Crowds," p. 653.

† The Fire Protection of Mills, p. 118.

TABLE III.—WEIGHTS OF MERCHANDISE.

Material.	Measurements.		Weights.		
	Floor space.	Cubic feet.	Gross.	Per sq. ft.	Per cu. ft.
Wool.					
Bale East India	3.0	12.0	340	113	28
" Australia	5.8	26.0	385	66	15
" South America	7.0	34.0	1000	143	29
" Oregon	6.9	33.0	482	70	15
" California	7.5	33.0	550	73	17
Bag wool	5.0	30.0	200	40	7
Stack of scoured wool	—	—	—	—	5
Woollen Goods.					
Case flannels	5.5	12.7	220	40	17
" flannels, heavy	7.1	15.2	330	46	22
" dress goods	5.5	22.0	460	84	21
" cassimeres	10.5	28.0	550	52	20
" underwear	7.3	21.0	350	48	16
" blankets	10.3	35.0	450	44	13
" horse-blankets	4.0	14.0	250	63	18
Cotton, etc.					
Bale	8.1	44.2	515	64	12
" compressed	4.1	21.6	550	134	25
" American Cotton Co.	4.0	11.0	263	66	24
" Planters' Compressed Co.	2.3	7.2	254	110	35
" jute	2.4	9.9	300	125	30
" jute lashings	2.6	10.5	450	172	43
" manila	3.2	10.9	280	88	26
" hemp	8.7	34.7	700	81	20
" sisal	5.3	17.0	400	75	24
Cotton Goods.					
Bale unbleached jeans	4.0	12.5	300	72	24
Piece duck	1.1	2.3	75	68	33
Bale brown sheetings	3.6	10.1	235	65	23
Case bleached sheetings	4.8	11.4	330	69	30
" quilts	7.2	19.0	295	41	16
Bale print cloth	4.0	9.3	175	44	19
Case prints	4.5	13.4	420	93	31
Bale tickings	3.3	8.8	325	99	37
Skeins cotton yarn	—	—	—	—	11
Burlaps	—	—	130	—	30
Jute bagging	1.4	5.3	100	70	24
Rags in Bales.					
White linen	8.5	39.5	910	107	23
White cotton	9.2	40.0	715	78	18
Brown cotton	7.6	30.0	442	59	15
Paper shavings	7.5	34.0	507	68	15
Sacking	16.0	65.0	450	28	7
Woollen	7.5	30.0	600	80	20
Jute butts	2.8	11.1	400	143	36
Paper.					
Calendered book	—	—	—	—	50
Supercalendered book	—	—	—	—	69
Newspaper	—	—	—	—	38
Straw board	—	—	—	—	33
Leather board	—	—	—	—	59
Writing	—	—	—	—	64
Wrapping	—	—	—	—	10
Manila	—	—	—	—	37

TABLE III.—WEIGHTS OF MERCHANDISE (*continued*).

Material.	Measurements.		Weights.		
	Floor space	Cubic feet	Gross.	Per sq. ft.	Per cu. ft.
GRAIN.*					
Wheat in bags	4.2	4.2	165	39	39
" in bulk	—	—	—	—	44
" " "	—	—	—	—	39
" " " mean	—	—	—	—	41
Barrels flour on side	4.1	5.4	218	53	40
" " on end	3.1	7.1	218	70	31
Corn in bags	3.6	3.6	112	31	31
Cornmeal in barrels	3.7	5.9	218	59	37
Oats in bags	3.3	3.6	96	29	27
Bale of hay	5.0	20.0	284	57	14
Hay, Dederick compressed	1.75	5.25	125	72	24
Straw " "	1.75	5.25	100	57	19
Tow " "	1.75	5.25	150	86	29
Excelsior " "	1.75	5.25	100	57	19
Hay, loose	—	—	—	—	4
DYESTUFFS, ETC.					
Hogshead bleaching powder	11.8	39.2	1200	102	31
" soda ash	10.8	29.2	1800	167	62
Box indigo	3.0	9.0	385	128	43
" cutch	4.0	3.3	150	38	45
" sumac	1.6	4.1	160	100	39
Caustic soda in iron drum	4.3	6.8	600	140	88
Barrel starch	3.0	10.5	250	83	23
" pearl alum	3.0	10.5	350	117	33
Box extract logwood	1.06	.8	55	52	70
Barrel lime	3.6	4.5	225	63	50
" cement, American	3.8	5.5	325	86	59
" " English	3.8	5.5	400	105	73
" plaster	3.7	6.1	325	88	53
" rosin	3.0	9.0	430	143	48
" lard oil	4.3	12.3	422	98	34
Rope	—	—	—	—	42
MISCELLANEOUS.					
Box tin	2.7	0.5	139	99	278
" glass	—	—	—	—	60
Crate crockery	9.9	39.6	1600	162	40
Cask crockery	13.4	42.5	600	52	14
Bale leather	7.3	12.2	190	26	16
" goatskins	11.2	16.7	300	27	18
" raw hides	6.0	30.0	400	67	13
" " " compressed	6.0	30.0	700	117	23
" sole leather	12.6	8.9	200	22	16
Pile sole leather	—	—	—	—	17
Barrel granulated sugar	3.0	7.5	317	106	42
" brown sugar	3.0	7.5	340	113	45
Cheese	—	—	—	—	30

and floor area supported by the different beams and girders. The second step is to determine approximately the weight of the floor and ceiling, and decide upon what superimposed load (per square foot) the floor shall be proportioned to carry.

Having done this, the next step will be to compute the required dimensions of the common floor joists.

* For pressure of grain in deep bins, see *Engineering News* of March 10, 1904, pp. 224 and 336, also of Dec. 15, 1904.

For most buildings the size of floor joist required can be readily determined by reference to Tables VI.–X. of this chapter.

For other floor loads the size of the common joists may be computed as follows: *Compute the load to be supported by a single joist and then, by the rules or tables in Chapter XVI. or XVIII., determine the dimensions of the joists to support the load.* (See Example 1.)

For the floors of all buildings except stores and warehouses the author recommends that the size of the common joists be determined by the rules or tables in Chapter XVIII. For stores and warehouses the size of the joists may be proportioned by the formulas for strength, Chapter XVI.

The dimensions of all special beams, such as headers, trimmers, beams supporting partitions, and also of the girders, should be found in the same way, viz., by computing the maximum load which the beam may have to support, and then the dimensions of a beam that will sustain the load with safety.

The manner of making the computations can be best explained by means of examples.

EXAMPLE 1.—The simplest floor framing is that shown in Fig. 1, in which all of the joists are of the same span and sustain

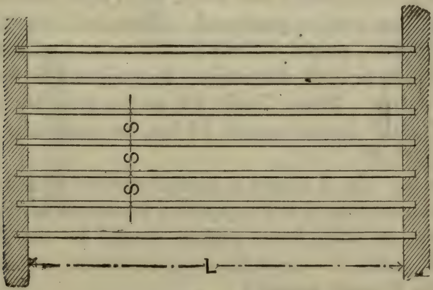


Fig. 1.
Plan of Floor Beams.

equal floor areas. In such a floor, the *floor area* supported by each joist is equal to the span L multiplied by the spacing S (in feet).

The *load* on each joist is equal to the *floor area* multiplied by the sum of the dead and superimposed loads. To show the application of the above rules and directions we will assume that Fig. 1 represents the framing of a floor in a dwelling- or lodging-

house, that $L=18$ ft., $S=16$ ins. or $1\frac{1}{3}$ ft., and that the timber is to be common white pine. The joists are to sustain a plastered ceiling and a double floor of $\frac{7}{8}$-inch boards. What should be the size of the joists?

Ans. The *floor area* supported by each joist will be $1\frac{1}{3}\times 18$, or 24 sq. feet. As the joists will probably have to be at least $2''\times 12''$, their weight will be about $4\frac{1}{2}$ lbs. per square foot (see Table I.). The plastered ceiling will weigh about 10 lbs. and the flooring 6 lbs., making the total weight of the floor $20\frac{1}{2}$ lbs. per sq. ft. For the superimposed load we should allow 40 lbs. (see p. 654). The load on a single joist will therefore be $60\frac{1}{2}$ lbs. $\times$ 24 sq. ft., or 1452 lbs.

From Table V., Chapter XVIII., we find that the maximum load for a 1×12 beam of 18 ft. span is 700 lbs., hence to support 1452 lbs. will require a breadth equal to $1452 \div 700$ or $2\frac{1}{16}$ ins. Therefore, to comply with the requirements for stiffness, the joists should be $2\frac{1}{16}''\times 12''$.

If we do not mind the deflection we can use the table for strength (Table VII., Chapter XVI.), which gives 960 lbs. for the safe load of a 1×12 beam, and dividing 1452 by 960 we have 1.51 for the required breadth of the beam; therefore a $1\frac{5}{8}''\times 12''$ joist will be strong enough, but would bend more than is desirable where the ceiling is to be plastered. Joists full $2''\times 12''$, spaced 16 ins. on centres, would answer in this case, but if they come $\frac{1}{4}$ in. scant in one or both dimensions they should be spaced only 12 ins. on centres. From Table VI. we see that the maximum span for $2''\times 12''$ joists, spaced 16 ins. on centres, in dwellings is given as 17′ 3″.

Example 2.—Fig. 2 shows a partial section of a dwelling, in which the second-floor joists support a plastered partition which also supports an attic floor. What should be the size of the second-floor joists to meet the requirements of *strength*, the timber to be Eastern spruce?

Note.—As the effect of a concentrated load in producing deflection, compared with a distributed load, is not as great as the comparative effect to produce rupture, whenever beams have a considerable *concentrated* load they may be calculated by the formula or tables *for strength only.*

Ans. The first step will be to determine the load on a single floor joist. We will assume that the joists are to be $2''\times 10''$, 12 ins. on centres, that both the first- and second-story ceilings are to be plastered, and that only single flooring will be used in

the second story and attic. We will assume that the attic joists are to be $2'' \times 8''$, 16 ins. centre to centre, and that the width of floor supported by the partition is 10 ft.

The second-floor area supported by a single joist will be $12'' \times$ 15 ft., or 15 sq. feet. The weight of the floor joists per sq. ft. will be 5 lbs., of the plastered ceiling 10 lbs., and of the flooring 3 lbs., making the dead load per sq. ft. 18 lbs. For the live or superimposed load we should allow 40 lbs., hence the load

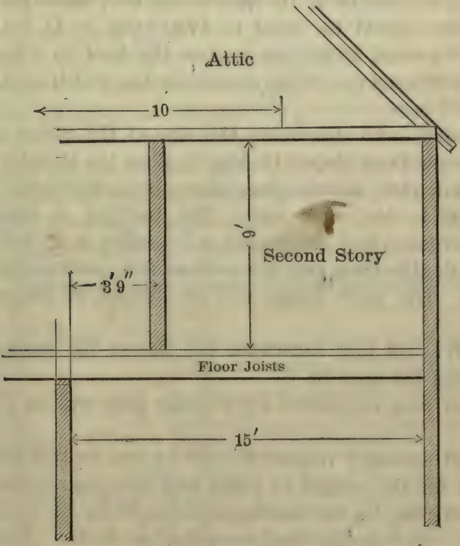


Fig. 2.
Section.

per sq. ft. on the second-floor joists due to the second floor and its load will be 58 lbs. As the floor area is 15 sq. ft. the load from the second floor will be 15×58 or 871 lbs. We must now find what will be the load from the partition and attic floor. The attic floor and ceiling will weigh about 16 lbs. per sq. ft., and 24 lbs. will be a sufficient allowance for the live load. The weight per lineal foot on the partition will therefore be 400 lbs. A partition of 2×4 studding lathed and plastered both sides will weigh about 20 lbs. per sq. ft.; hence the partition itself will

weigh 180 lbs. per lineal ft. The partition and attic floor will therefore bring a load of 580 lbs. on each second-floor joist, *concentrated* at a point *one-fourth* of the span from the inner end of the joist. To combine this concentrated load with the load from second floor, we must multiply the concentrated load by 1.5 (see page 565), which gives an equivalent distributed load of 870 lbs. Adding this to the second-floor load we have 1740 lbs. as the total load for which each joist should be proportioned. From Table VI., Chapter XVI., we find that the safe load for a 1×10 spruce beam of 15 ft. span is 933 lbs.; hence the breadth of the joists should be equal to $1740 \div 933$ or $1\frac{7}{8}$ ins. If the joists were spaced 16 ins. on centres the load on a single joist would be increased one-third, or to 2320 lbs., which would require a $1\frac{3}{4}'' \times 12''$ joist.

Example 3.—To determine the size of the girder and floor timbers in the floor shown in Fig. 3, all of the timbers being of Texas yellow pine, and the floor above being supported by posts and girders in the same way. The building is intended for lodging purposes, and the height of the story is 10 feet. There is to be a double floor and the ceilings and partitions are to be plastered. The floor joists will be spaced 16 ins. centre to centre.

Ans. We will first determine the size of the common joists at *A*, calling the span 24 ft.

The floor area supported by a single joist will be $24 \times 1\frac{1}{3}$, or 32 sq. ft.

As it will probably require $2'' \times 14''$ joists, we will allow 7 lbs. per sq. ft. for the weight of joists and bridging, 10 lbs. for the ceiling, and 6 lbs. for the flooring, making 23 lbs. per sq. ft. for the dead load. For the live load we will allow 40 lbs. The load for which the joists should be proportioned will therefore be 32×63, or 2016 lbs. As the stiffness of Texas pine is nearly the same as that of Georgia yellow pine, we may use Table II., Chapter XVIII., to find the maximum load for a $1'' \times 14''$ beam of 24 ft. span. The load given in the table is 1042 lbs., hence the thickness of the joists must equal $2016 \div 1042$ or 2 ins. Therefore $2'' \times 14''$ joists, 16 ins. on centres should be used, but they should run full 2 ins. thick.

The joists at *B* have to support a partition, but as the span is much less, and the partition is quite near the end of the joists, it will be safe to make them of the same size as at *A*.

The joists at C have the same floor load to support as at A, and in addition the weight of the partition, which is concentrated one-third of the span from one support. As the partition is 10 ft. high, $13\frac{1}{3}$ sq. ft. of partition will be supported by each joist

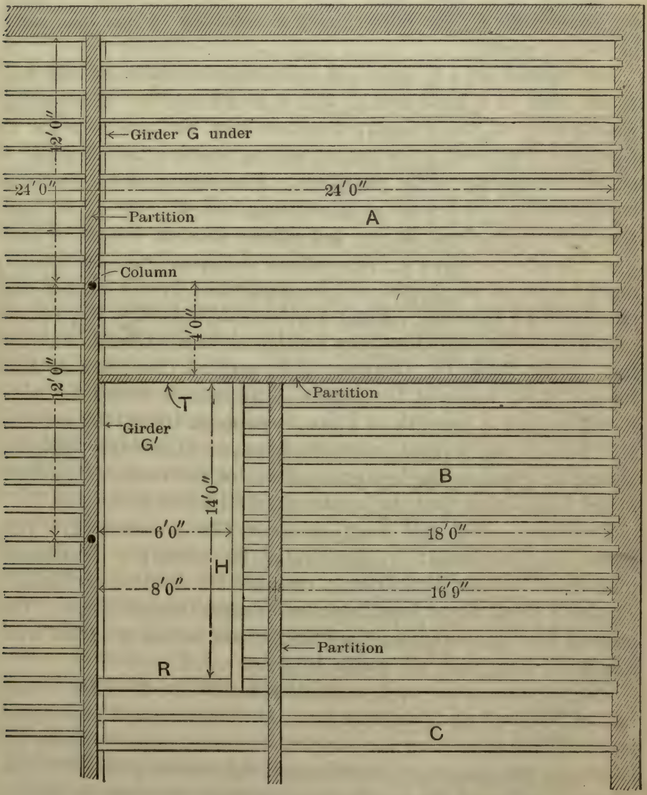


Fig. 3.
Plan of Floor Framing showing Partitions Above.

(the joists being 16 ins. on centres). Assuming 20 lbs. per sq. ft. as the weight of the partition, we have 267 lbs. as the weight from the partition to be borne by each joist. To reduce this to

an equivalent distributed load, we should multiply by 1.78, which gives 468 lbs. The joist at *C*, therefore, should be proportioned to a uniformly distributed load of 2016+468 or 2484 lbs., which will require a 14-inch joist 2.4 ins. thick, or say $2\frac{1}{2}\times 14$.

Header.—We will next determine the required breadth for the header *H*, the depth being necessarily 14 ins. (the same as for the joists).

The header is 14 ft. long and must support the floor half way to the wall, or a floor area of 14×9, or 126 sq. ft. Multiplying this area by 63, the weight per square foot, we have 7938 lbs. as the total floor load to be supported, to which must be added a certain percentage of the partition. The portion of the partition supported by the header is 12′–8″ long (14′–0″—1′–4′) 10′ high and will weigh about 20 lbs. per square foot, or a total of 2532 lbs. As the partition is one-ninth of the span from the header, eight-ninths of its weight will be supported by the header and one-ninth by the wall. Eight-ninths of 2532 is 2251 lbs., which added to the floor load makes a total load for the header of 10,188 lbs. From Table IV., Chapter XVI., we find that the safe load for a $1''\times 14''$ beam of Texas pine, 14 ft. span is 2520 lbs., hence it will require a breadth of 4 ins. to support 10,630 lbs. If the tail beams are framed into the header, additional thickness should be given to the header to allow for the weakening effects of the framing, so that the header should be at least $5''\times 14''$.

Trimmers.—We will next consider the trimmer *T*. This beam has four loads: A distributed floor load; a distributed load from the partition above; one-half the load on the header *H*, and a small direct load from the longitudinal partition. The strip of floor supported by the trimmer will be about 12 ins. wide and 24 ft. long, and will weigh 1512 lbs.

The partition above will weigh $10\times 24\times 20$, or 4800 lbs. One-half of the load on *H* is 5094 lbs. As this is concentrated one-fourth of the span from the support, we must multiply it by 1.5 to obtain the equivalent distributed load, which gives 7641 lbs. About 8 inches of the longitudinal partition must be supported by the trimmer, and this will weigh 133 lbs. As it is concentrated one-third of the span from the support, we must multiply by 1.78 to obtain the equivalent distributed load—which gives 236 lbs.

The total load for which the trimmer must be computed will therefore be:

From the floor	1512
From the partition above	4800
From the header	7641
From the longitudinal partition	236
Total	14,189

The trimmer should be of the same depth as the joists, 14 ins. From Table IV., Chapter XVI., we find that a 1×14 in. Texas pine beam of 24 ft. span will safely support 1470 lbs.; hence the breadth of the trimmer must $= 14{,}189 \div 1470 = 9.5$ ins., and the header should be hung in a stirrup or joist hanger. The load on the trimmer R will be the same as on the trimmer T, except for the cross partition. Deducting the weight of this partition, we have 9389 pounds for the equivalent distributed load on R, which will require a breadth $= 9389 \div 1470 = 6.4$ ins.

Girders.—The floor area supported by girder G is equal to 12×24 ft., or 288 square feet. As a general rule, it will be safe in estimating the live load on girders to take only 85 per cent. of the load assumed for the floor beams, because there will always be some portion of the floor supported by the girder that is not loaded, and probably other portions that will not be loaded up to the assumed load. 85 per cent. of 40 pounds is 34 pounds. The dead load of the floor and ceiling will be about 23 lbs., and the girder itself will weigh at least 1 pound per sq. ft. of floor more, so that we will use 58 lbs. per square ft. for the total floor load on girders. As girder G supports 288 sq. ft., this will be equivalent to 16,704 lbs. The girder also supports a partition, 9′ high above, which will weigh $12 \times 9 \times 20 = 2160$ lbs. The total load for which the girder should be proportioned is therefore 18,864 lbs. Assuming 12 ins. for the depth of the girder, we find from Table IV., Chapter XVI., that the safe load for a $1 + 12$ beam of 12 ft. span is 2160 lbs., hence the breadth of the girder should be $18{,}864 \div 2160 = 9$ ins.

The girder G' supports a floor area at the left of $12 \times 12 = 144$ sq. ft., which represents a distributed load of 8352 lbs. On the right side of the girder there is a strip of floor 40 ins. wide by 12 ft. long (8 ins. of the floor being included in the load on T) which will weigh 2320 lbs. This may be considered as a concentrated load applied 20 ins. or one-seventh of the span from the end of the girder, in which case the effect of the load is practically the same as if the load were distributed.

The load coming upon the girder from T will equal one-half of the actual distributed load on T, plus $\frac{3}{8}$ ($\frac{1}{2}$ of $\frac{3}{4}$) of the load on H.

The load on H we found to be 10,188 lbs., and three-eighths is 3820 lbs. The actual distributed load on T we found to be 6312 lbs., and one-half of this is 3156 lbs. Hence the trimmer T transmits a load of 6976 lbs. to the girder, which must be considered as a concentrated load applied at one-third of the span from the support, and hence we must multiply it by 1.78 to obtain the equivalent distributed load, which gives 12,417 lbs.

The load for which the girder G' should be computed will therefore be

From the floor at the left	8,352	lbs.
From the floor at the right	2,320	"
From the trimmer T	12,417	"
From the partition above	2,160	"
Total	25,249	"

This will require a beam 11.7 ins. wide. For this floor, therefore, we will require a $10'' \times 12''$ girder at G, a $12'' \times 12''$ at G', a $9'' \times 14''$ beam for the trimmer T, $6\frac{1}{2}'' \times 14''$ for R, $5'' \times 14''$ for H, and $2'' \times 14''$ joists at A and B, and $2\frac{1}{2}'' \times 14''$ joists at C. This example illustrates nearly all of the computations that are required to determine the size of the joists and special beams in any ordinary floor construction.

The method of computation is the same for any floor load, the only difference being that the greater the live load assumed the greater will be the loads for which the beams must be proportioned.

As will be seen, the most laborious computations are those for beams which receive loads from different sources, and it will generally be found that the weakest portions of any particular floor are the headers, trimmers, and girders, and the beams which support partitions.

Strength of Mill Floors.—The beams and girders for mill floors should be computed by the same process as exemplified in the foregoing examples, viz., first determining the load on the beam and then the size of timber required to support it.

Required Thickness of Plank Flooring.—The thickness of the plank flooring in mill construction may be determined by formulas (*a*) and (*b*), following:

$$\left.\begin{array}{r}\text{Thickness of plank in ins.}\\ \text{required for strength}\end{array}\right\} = \sqrt{\frac{\text{weight per sq. ft.}\times L^2}{24\times A}} \qquad (a)$$

$$\left.\begin{array}{r}\text{Thickness of plank in ins.}\\ \text{required for stiffness}\end{array}\right\} = \sqrt[3]{\frac{\text{weight per sq. ft.}\times L^3}{19.2\times e_1}} \qquad (b)$$

L denotes span in feet, centre to centre of beams, A the constants for strength, p. 567, and e_1 the constant for stiffness (p. 595).

When the planks are connected by ¾-in. splines, and extend over two spans, formula (*a*) may be used. If the planks are in single lengths from beam to beam, or are not splined, then formula (*b*) should be used.

Table IV. shows the safe loads for plank flooring of different thicknesses and spans, as derived from the formulas for strength and stiffness, the plain figures denoting the loads given by the formula for strength and the figures in italics those given by the formula for stiffness.

The span is supposed to be measured from centre to centre of beams. The plain figures should be considered safe only for splined floors and where the planks are continuous over at least two spans. If the thickness of the plank falls short one-fourth or even one-eighth inch from the dimensions given, the safe loads must be materially reduced.

Tables for the Maximum Span of Floor Joists.—As the timbers commonly used for floor joists are sawn to regular sizes, and are usually spaced either 12 or 16 ins. centre to centre it is practicable to show by means of tables the size of joist required to support a given load with a given span and spacing. After having computed various tables the author has found that tables giving the *maximum safe span* are the most convenient for general use, and the following tables have accordingly been prepared, which show at a glance the maximum span for which different sizes of floor and ceiling joists should be used for different loads and spacings; it is believed that they will be found applicable to most buildings in which wooden floor joists are used.

By knowing the size of the room to be covered and the purpose for which it is to be used, the size of joist required can be told at a

TABLE IV.—SAFE LIVE LOAD IN POUNDS PER SQUARE FOOT FOR PLANK FLOORING.*

(See explanation on preceding page.)

LONG-LEAF YELLOW PINE.

Thickness of plank in ins.	Distance between centres of floor beams in feet.								
	4	5	6	7	8	9	10	11	12
1⅞	515	325	222	160	120	92	72		
	258	*126*	*68*	*38*	*21*	*11*	*5*		
2⅜	831	527	362	262	197	153	121	97	80
	536	*268*	*149*	*88*	*54*	*34*	*24*	*12*	*6*
2¾	1118	710	488	354	267	208	165	134	110
	838	*421*	*237*	*144*	*91*	*59*	*38*	*25*	*15*
3½		1158	798	582	442	345	276	225	186
		884	*504*	*310*	*202*	*136*	*94*	*67*	*47*
4			1046	763	580	454	364	296	246
			759	*470*	*308*	*210*	*148*	*106*	*77*
5				1200	913	716	576	471	392
				934	*618*	*427*	*304*	*223*	*166*
6					1322	1038	836	686	572
					1081	*751*	*540*	*398*	*300*

OREGON PINE OR SHORT-LEAF YELLOW PINE.

Thickness of plank in ins.	Distance centre to centre of floor beams in feet.								
	4	5	6	7	8	9	10	11	12
1⅞	462	291	199	143	106	81	64		
	205	*99*	*52*	*28*	*15*	*7*			
2⅜	747	473	324	234	176	136	107		
	428	*212*	*117*	*68*	*41*	*25*	*14*		
2¾	1005	637	438	317	239	185	147	119	97
	670	*335*	*187*	*112*	*69*	*44*	*28*	*17*	*9*
3½		1040	717	522	395	308	246	200	165
		706	*401*	*246*	*159*	*106*	*72*	*50*	*34*
4		1362	940	685	520	406	325	265	220
		1061	*606*	*374*	*244*	*165*	*115*	*81*	*58*
5			1476	1078	819	642	516	422	351
			1198	*745*	*491*	*338*	*240*	*174*	*128*
6				1560	1187	932	749	614	512
				1302	*863*	*597*	*428*	*314*	*236*

* Weight of ceiling, if any, to be deducted.

TABLE IV.—SAFE LIVE LOAD IN POUNDS PER SQUARE FOOT FOR PLANK FLOORING* (*continued*).

(See explanation on page 667.)

SPRUCE.

Thickness of plank in ins.	Distance between centres of floor beams in feet.								
	4	5	6	7	8	9	10	11	12
1⅞	360	227	155	111	83	64	50		
	188	*92*	*49*	*28*	*15*	*8*			
2⅜	581	368	252	182	137	105	83	67	54
	391	*194*	*108*	*64*	*39*	*24*	*15*		
2¾	782	496	341	247	186	144	115	93	76
	612	*307*	*173*	*104*	*66*	*42*	*28*	*18*	
3½	1228	781	548	391	296	231	184	150	124
	1274	*644*	*367*	*225*	*146*	*98*	*68*	*47*	*33*
4		1060	731	533	405	317	253	207	171
		968	*554*	*343*	*225*	*153*	*108*	*77*	*56*
5			1148	839	638	500	402	329	273
			1093	*682*	*450*	*311*	*212*	*162*	*120*
6				1213	924	725	583	478	400
				1188	*789*	*548*	*394*	*290*	*220*

WHITE PINE.

Thickness of plank in ins.	Distance between centres of floor beams in feet.								
	4	5	6	7	8	9	10	11	12
1⅞	307	193	131	94	70	53	41		
	153	*74*	*39*	*21*	*11*	*5*			
2⅜	496	314	114	154	116	89	70	56	
	318	*157*	*86*	*50*	*40*	*18*	*10*		
2¾	668	424	290	210	158	122	97	78	63
	499	*249*	*139*	*83*	*52*	*33*	*20*	*12*	
3½	1088	691	476	316	261	203	162	131	108
	1041	*526*	*298*	*183*	*119*	*78*	*53*	*36*	*25*
4		906	625	455	345	269	215	175	145
		791	*451*	*278*	*181*	*123*	*85*	*60*	*43*
5			982	716	544	426	342	281	232
			893	*555*	*366*	*251*	*178*	*129*	*95*
6			1419	1037	789	619	497	407	339
			1553	*970*	*643*	*445*	*319*	*234*	*175*

* Weight of ceiling, if any, to be deducted.

glance. Incidentally the tables also show which kind of wood will be most economical.

If, owing to the room being irregular in shape, the joists must be of different lengths, the spacing or thickness of the joists may be varied, so that the same depth may be used throughout.

The only *precautions* to be exercised in using these tables are in regard to the superimposed load and the *actual size* of the timbers.

The total loads for which the maximum spans have been computed are given at the head of each table. The actual weight of the floor (beams, flooring, plastering, and deafening, if any) subtracted from the total load will give the superimposed load, *i.e.*, the load which the floor is expected to carry.

If the joists do not run full to dimensions, the span or spacing must be reduced from that given in the tables, and as in certain localities the stock sizes of joists often run from $\frac{1}{4}$ inch to $\frac{3}{8}$ inch scant of the nominal dimensions, this fact should always be taken into account when determining upon the size of joists. In this connection it will be convenient to remember that a 2-inch joist spaced 16 ins. c. to c. has the same strength as a $1\frac{1}{2}$-inch joist 12 ins. centre to centre.

A reduction should also be made for any cutting of the joists that may be required.

No allowance has been made for partitions, and when they are to be supported by the floor joists additional joists should be used or the span reduced according to the relative direction or position of the partition and joists.

Tables V. to IX. inclusive, were computed by the formula for stiffness, on the assumption that the deflection should not exceed $\frac{1}{30}$ of an inch per foot of span. Tables X. and XI. were computed by the formula for strength.

The spans given in these tables come within the requirements of the Buffalo and Denver building laws, and Tables V., VII., VIII., IX., and X. comply with the Chicago law and very nearly with the New York law, but to comply with the Boston law a reduction of about one-sixth must be made from the spans given.

By Georgia pine is meant the long-leaf yellow or hard pine.

TABLE V.—MAXIMUM SPAN FOR CEILING JOISTS.

Total load, 20 pounds per square foot.

Size of Joist.	Dist. on Centres.	Hemlock		White Pine.		Spruce or Norway Pine.		Oregon or Texas Pine.		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
2×4	12	9	3	9	5	10	1	10	5	11	2
2×4	16	8	5	8	6	9	1	9	5	10	1
2×6	12	14	0	14	1	15	1	15	7	16	8
2×6	16	12	8	12	10	13	8	14	2	15	2
2×8	12	18	8	18	10	20	1	20	9	22	4
2×8	16	17	0	17	2	18	4	18	11	20	5
2×8	20	15	9	15	10	17	0	17	6	18	10
Total load, 24 pounds per square foot.											
2×10	12	22	0	22	2	23	8	24	5	26	4
2×10	16	20	0	20	2	21	7	22	3	23	10
2×10	20	18	6	18	8	20	0	20	7	22	2
2×12	12	26	5	26	8	28	5	29	4	31	7
2×12	16	24	0	24	2	25	10	26	8	28	8
2×12	20	22	3	22	5	24	0	24	8	26	8

See remarks, page 670.

TABLE VI.—MAXIMUM SPAN FOR FLOOR JOISTS.

DWELLINGS, TENEMENTS, AND GRAMMAR-SCHOOL ROOMS WITH FIXED DESKS.

Total load, 60 pounds per square foot.

Size of Joists.	Dist. on Centres.	Hemlock		White Pine.		Spruce or Norway Pine.		Oregon or Texas Pine.		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
2×6	12	9	9	9	10	10	5	10	10	11	7
2×6	16	8	9	8	10	9	6	9	10	10	6
3×6	12	11	1	11	2	12	0	12	5	13	4
3×6	16	10	1	10	2	10	10	11	2	12	1
2×8	12	12	11	13	1	13	11	14	5	15	6
2×8	16	11	9	11	10	12	8	13	1	14	1
3×8	12	14	9	14	11	16	0	16	6	17	8
3×8	16	13	6	13	7	14	6	15	0	16	2
2×10	12	16	2	16	4	17	5	18	0	19	4
2×10	16	14	9	14	10	15	9	16	4	17	7
Total load, 66 pounds per square foot.											
3×10	12	18	0	18	1	19	3	20	0	21	6
3×10	16	16	3	16	5	17	7	18	2	19	6
2×12	12	18	10	19	0	20	3	20	10	22	6
2×12	16	17	2	17	3	18	4	19	0	20	6
3×12	12	21	6	21	8	23	2	24	0	25	9
3×12	16	19	7	19	8	21	1	21	9	23	5
2×14	12	22	0	22	2	23	8	24	4	26	3
2×14	16	20	0	20	1	21	6	22	2	23	10
2½×14	12	23	8	23	10	25	6	26	3	28	3
2½×14	16	21	6	21	8	23	2	23	10	25	8
3×14	12	25	4	25	4	27	1	28	0	30	1
3×14	16	23	0	23	0	24	7	25	4	27	4

TABLE VII.—MAXIMUM SPAN FOR FLOOR JOISTS.

OFFICE BUILDINGS.

Total load, 93 pounds per square foot.

Size of Joists.	Dist. on Centres.	White Pine		Spruce or Norway Pine.		Oregon or Texas Pine		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
3×8	12	12	10	13	9	14	2	15	4
3×8	16	11	8	12	6	12	10	13	10
2×10	12	14	1	15	1	15	6	16	7
2×10	16	12	9	13	8	14	1	15	2
3×10	12	16	1	17	3	17	9	19	2
3×10	16	14	8	15	8	16	2	17	5
2×12	12	16	10	18	1	18	8	20	1
2×12	16	15	4	16	5	17	0	18	3
Total load, 96 pounds per square foot.									
3×12	12	19	2	20	6	21	2	22	9
3×12	16	17	5	18	7	19	3	20	8
2×14	12	19	6	20	10	21	7	23	2
2×14	16	17	9	19	0	19	7	21	2
2½×14	12	21	1	22	6	23	2	25	0
2½×14	16	19	2	20	4	21	2	22	8
3×14	12	22	4	23	10	24	8	27	7
3×14	16	20	4	21	8	22	5	24	1

See remarks, page 670.

TABLE VIII.—MAXIMUM SPAN FOR FLOOR JOISTS.

CHURCHES AND THEATRES WITH FIXED SEATS.

Total load, 102 pounds per square foot.

Size of Joists.	Dist. on Centres.	White Pine		Spruce or Norway Pine.		Oregon or Texas Pine		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
3×8	12	12	6	13	4	13	9	14	10
3×8	16	11	4	12	2	12	6	13	6
2×10	12	13	7	14	7	15	1	16	2
2×10	16	12	4	13	3	13	8	14	9
3×10	12	15	8	16	9	17	3	18	7
3×10	16	14	2	15	2	15	8	16	10
2×12	12	16	5	17	7	18	1	19	6
2×12	16	14	10	15	11	16	5	17	8
Total load, 105 pounds per square foot.									
3×12	12	18	7	19	11	20	6	22	1
3×12	16	16	10	18	1	18	7	20	1
2×14	12	19	0	20	3	20	10	22	6
2×14	16	17	3	18	5	19	0	20	6
2½×14	12	20	4	21	9	22	6	24	3
2½×14	16	18	7	19	10	20	6	22	1
3×14	12	21	8	23	2	23	10	25	9
3×14	16	19	8	21	1	21	9	23	4

TABLE IX.—MAXIMUM SPAN FOR FLOOR JOISTS.

ASSEMBLY HALLS AND CORRIDORS.

Total load, 123 pounds per square foot.

Size of Joists.	Dist. on Centres.	White Pine		Spruce or Norway Pine.		Oregon or Texas Pine		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
3×8	12	11	7	12	7	13	0	14	0
3×8	16	10	8	11	4	11	9	12	8
2×10	12	12	10	13	9	14	2	15	2
2×10	16	11	7	12	6	12	10	13	10
3×10	12	14	8	15	8	16	2	17	5
3×10	16	13	4	14	3	14	9	15	10
2×12	12	15	4	16	6	17	0	18	3
2×12	16	14	0	15	0	15	5	16	7
Total load, 126 pounds per square foot.									
3×12	12	17	6	18	8	19	3	20	9
3×12	16	15	10	17	0	17	7	18	11
2×14	12	17	10	19	1	19	8	21	2
2×14	16	16	2	17	4	17	11	19	3
2½×14	12	19	3	20	6	21	2	22	9
2½×14	16	17	6	18	8	19	3	20	9
3×14	12	20	5	21	9	22	6	24	3
3×14	16	18	7	19	10	20	6	22	1

See remarks, page 670.

TABLE X.—MAXIMUM SPAN FOR FLOOR JOISTS.

RETAIL STORES.

Total load, 174 pounds per square foot.

Size of Joists.	Dist. on Centres.	White Pine.		Spruce or Norway Pine.		Oregon or Texas Pine.		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
3×8	12	11	6	12	5	14	1	14	9
3×8	16	9	11	10	2	12	2	12	9
2×10	12	11	8	12	8	14	5	15	1
2×10	16	10	2	10	11	12	5	13	1
3×10	12	14	4	15	6	17	7	18	7
3×10	16	12	5	13	5	15	2	16	0
2×12	12	14	1	15	2	17	2	18	2
2×12	16	12	2	13	1	14	11	15	8
Total load, 177 pounds per square foot.									
3×12	12	17	2	18	5	20	11	22	1
3×12	16	14	10	16	0	18	2	19	1
2×14	12	16	3	17	7	19	11	21	1
2×14	16	14	2	15	2	17	3	18	2
2½×14	12	18	2	19	7	22	3	23	6
2½×14	16	15	9	17	0	19	3	20	4
3×14	12	19	11	21	6	24	5	25	8
3×14	16	17	3	18	7	21	2	22	3

TABLE XI.—MAXIMUM SPAN FOR RAFTERS.

A. SHINGLED ROOFS NOT PLASTERED.*

Total load, 48 pounds per square foot.

Size of Joists.	Dist. on Centre.	Hemlock		White Pine.		Spruce or Norway Pine.		Oregon or Texas Pine.		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
2×4	16	7	4	7	9	8	4	9	6	10	10
2×4	20	6	7	6	10	7	6	8	6	8	10
2×6	16	11	1	11	7	12	6	14	2	15	0
2×6	20	9	11	10	4	11	2	12	8	13	4
3×6	16	13	7	14	2	15	3	17	5	18	3
3×6	20	12	2	12	8	13	8	15	7	16	4
2×8	16	14	9	15	6	16	8	18	11	20	0
2×8	20	13	3	13	10	14	11	16	11	17	10
2×8	24	12	1	12	7	13	7	15	6	16	3
2×10	16	18	6	19	3	20	10	23	8	25	0
2×10	20	16	7	17	3	18	8	21	2	22	3
2×10	24	15	1	15	9	17	0	19	3	20	4

B. SLATE ROOFS NOT PLASTERED, OR SHINGLE ROOFS PLASTERED.*

Total load, 57 pounds per square foot.

Size of Joists.	Dist. on Centres.	Hemlock		White Pine.		Spruce.		Oregon Pine.		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
2×4	16	6	9	7	1	7	7	8	8	9	2
2×4	20	6	0	6	4	6	9	7	9	8	2
2×6	16	10	2	10	7	11	6	13	0	13	8
2×6	20	9	1	9	6	10	2	11	7	12	3
3×6	16	12	6	13	0	14	1	15	11	16	9
3×6	20	11	1	11	8	12	7	14	3	15	0
2×8	16	13	7	14	2	15	3	17	4	18	3
2×8	20	12	2	12	8	13	8	15	6	16	4
2×8	24	11	1	11	7	12	6	14	2	14	11
3×8	16	16	7	17	4	18	9	21	3	22	5
3×8	20	14	10	15	6	16	9	19	0	20	1
3×8	24	13	7	14	2	15	3	17	4	18	4
2×10	16	17	0	17	8	19	2	21	7	22	10
2×10	20	15	2	15	10	17	1	19	4	20	6
2×10	24	13	10	14	6	15	7	17	8	18	8

* These tables allow for a snowfall of 2 feet. In the Southern States the spans in section A will be safe for slate or gravel roofs, if the joists are full to dimensions.

TABLE XI.—MAXIMUM SPAN FOR RAFTERS (*continued*).

C. SLATE ROOFS PLASTERED, OR GRAVEL ROOFS NOT PLASTERED.*

Total load, 66 pounds per square foot.

Size of Joists.	Dist. on Centres.	Hemlock		White Pine.		Spruce or Norway Pine.		Oregon or Texas Pine.		Georgia Pine.	
	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.
2×6	16	9	5	9	10	10	8	12	1	12	9
2×6	20	8	6	8	10	9	6	10	9	11	5
3×6	16	11	7	12	1	13	1	14	10	15	7
3×6	20	10	4	10	10	11	8	13	3	14	0
2×8	16	12	7	13	2	14	2	16	2	17	0
2×Ͻ	20	11	3	11	9	12	9	14	5	15	2
2×8	24	10	3	10	9	11	7	13	2	13	10
3×8	16	15	5	16	1	17	5	19	9	20	10
3×8	20	13	9	14	5	15	3	17	8	18	8
3×8	24	12	7	13	2	14	2	16	2	17	0
2×10	16	15	9	16	6	17	9	20	2	21	3
2×10	20	14	1	14	8	15	11	18	0	19	0
2×10	24	12	10	13	5	14	6	16	6	17	5
2×12	16	18	10	19	9	21	4	24	2	25	6
2×12	20	16	10	17	8	19	1	21	8	22	10
2×12	24	15	5	16	1	17	5	19	9	20	10

* These tables are intended for climates where a snowfall of 2 feet may be expected. In the Southern States, where there is no snow to speak of, the spans in the first sections will be safe for slate or gravel roofs if the joists are sawn full to dimensions.

To Determine the Strength of an Existing Floor. —When a building is leased for mercantile or manufacturing purposes the tenant will generally desire to know the greatest load which it will be safe to put upon the floors, and some building laws require that the safe load for the floors in certain classes of buildings shall be computed and posted in a conspicuous place in each story. It is therefore important that every architect should know how to compute the safe strength of any existing floor.

The problem is practically the reverse of that of proportioning a floor to a given load.

In speaking of the strength of a floor a distinction should be made between the *safe strength* and the *safe load.* The "safe strength" should mean the maximum safe load for the beams, including the weight of the construction, flooring, and ceiling, while the "safe load" refers to the maximum load which may safely be placed upon the floor. The *safe* load is found by first computing the safe strength and then subtracting the weight of the materials forming the floor, including the ceiling below,

if there is one. The most convenient measurement for either the "safe strength" or the "safe load" of a floor is in pounds per square foot.

The following example will serve to show the process of determining the safe load for an ordinary warehouse floor.

EXAMPLE 4.—What is the safe load per square foot for a floor framed as shown in Fig. 4, all of the timber being Eastern spruce,

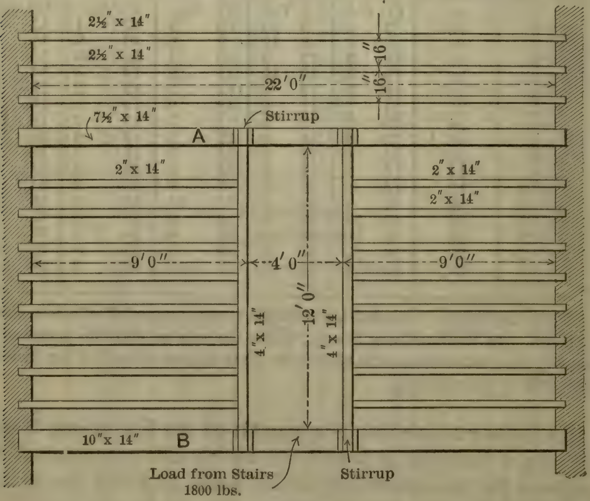


Fig. 4

the beams being covered with two thicknesses of $\frac{7}{8}$-inch flooring and having a corrugated iron ceiling below?

The first step will be to find the safe strength of the 22-ft. joists. As this is a warehouse floor we will use the tables for strength entirely. From Table VI., Chapter XVI., we find the safe strength of a 1×14 spruce beam of 22 ft. span to be 1,247 lbs., hence the strength of a $2\frac{1}{2}'' \times 14''$ beam will be $2\frac{1}{2} \times 1{,}247$, or 3,117 lbs. As the joists are 16 ins. on centres, each joist supports a floor area of $1\frac{1}{3} \times 22$ ft. $= 29\frac{1}{3}$ sq. ft. The *safe strength per square foot* of this portion of the floor will therefore be $3{,}117 \div 29.3$ or, 106 lbs. The weight of the floor per square foot will be about $6\frac{1}{2}$ lbs. for the joists, 6 lbs. for the flooring, and 1 lb. for the corrugated iron ceiling, or, say 14 lbs. in all. Therefore the *safe load* per square foot for the 22-ft. joists will be $106 - 14$, or 92 lbs.

We will next find the safe load for the 4×14 headers at each side

of the stair well. As the tail beams are framed into the headers, we should deduct one inch from the thickness of the beam for the loss of strength in framing, leaving $3'' \times 14''$ for the effective dimensions of the headers. From Table VI., Chapter XVI., we find the safe strength of a 1×14, 12-ft. span to be 2,286 lbs. Hence the strength of the 3×14 will be 6,858 lbs. The floor area supported by each header is $4\frac{1}{2} \times 12$ ft. $=54$ sq. ft.; hence the safe strength of the header per square foot of floor $=6,858 \div 54 =$ 127 lbs. Deducting the weight of the floor per sq. ft. 14 lbs., we have 113 lbs. per sq. ft. for the safe load.

Strength of Trimmer A. — Trimmer *A* supports about the same amount of flooring as one of the common joists, and also the ends of the headers. Deducting $2\frac{1}{2}$ ins., the thicknesses of the common joists, we have a $5'' \times 14''$ beam left to support the headers. As the headers are supported in iron stirrups no deduction in strength need be made for framing.

To find the safe strength of a beam loaded with two concentrated loads, equally distant from the supports, we must use formula 14, Chapter XVI. In this case $m=8'\ 10''$ or $8\frac{5}{6}'$, and $A=70$.

Applying the formula safe load at each point $= \dfrac{5 \times 196 \times 70}{4 \times 8\frac{5}{6}} =$ 1,942 lbs. The floor area supported by one stirrup is equal to one-half of the area supported by the header, or 27 sq. ft.; hence the safe strength per square foot of the 5×14 header is $1,942 \div 27$, or 72 lbs., and deducting 14 lbs. for weight of the floor, we have 58 lbs. per square foot as the safe load that the trimmer will support on the floor at each side of the stairs. Considering that the safe load for the $2\frac{1}{2}$ ins. which we deducted to take the place of a common joist is 92 lbs., we might place the safe load for the trimmer at an average of 92 and 58, or 75 lbs.

Trimmer B.—This timber has to support the same floor loads as trimmer *A*, and also the bottom of a flight of stairs for which an allowance of at least 1,800 lbs. should be made.

This stair load being practically concentrated at the centre of the trimmer is equivalent to a distributed load of 3,600 lbs. As the safe load for a 1×14-inch joist of 22 ft. span is 1,247 lbs., it will require a thickness $=3,600 \div 1,247$, or $2\frac{7}{8}$ ins., to support the stairs, leaving $7\frac{1}{8}$ ins. to support the floor loads. As this is $\frac{3}{8}$ in. less than the thickness of trimmer *A*, it is evident that the strength of the floor at *B* will be a little less than at *A*, but as it is improbable that the entire floor space will be loaded at any

given time, it would be safe to rate the strength of the floor at each side of the stairway at 75 lbs. per square foot, *live load,* and beyond the stairway at 92 lbs.

Partitions.—When the floor supports partitions their weight and any load resting upon them must be taken into account in determining the safe load for the floor. If the partition runs the same way as the joists, then only the joist directly under the partition, and the joists at each side will be affected; but if the partition runs across the joists, then it affects the safe load of the entire floor.

Example 5.—Suppose that the 22-ft. joists in the floor shown by Fig. 4 have to support a plastered partition 12 ft. high running across the joists half-way between the walls, what will be the safe load for the floor?

Ans. A plastered partition with $2'' \times 4''$ or $2'' \times 6''$ studding 16 ins. on centres will weigh about 20 lbs. per sq. ft.; hence a partition 12 ft. high will weigh 240 lbs. per lineal foot. As the joists are 16 ins. on centres, each joist will support $1\frac{1}{3}$ lineal ft. of partition weighing 320 lbs. As this load is concentrated at the centre of the joists it is equivalent to a distributed load of 640 lbs. In Example 4, we found the safe distributed load for a $2\frac{1}{2}'' \times 14''$ spruce joist of 22 ft. span to be 3,117. Subtracting 640 lbs. from this we have 2,477 lbs., which may be used for the floor. As the floor area supported by one joist is $29\frac{1}{3}$ sq. ft., the safe strength of the floor per sq. ft. will be $2{,}477 \div 29\frac{1}{3}$, or 84 lbs., and the safe load 70 lbs. Hence the partition decreases the safe load by 22 lbs. per square foot.

Whenever the upper-floor joists are supported by a partition carried by a floor below, the effect of the partition and its load upon the strength of the lower floor should be very carefully computed.

Bridging of Floor Beams.—By "bridging" is meant a system of bracing floor beams, either by means of small struts, as in Fig. 5, or by means of single pieces of boards set at right angles to the joists, and fitting in between them.

The effect of this bracing is decidedly beneficial in sustaining any *concentrated* weight upon a floor; but it does not materially strengthen a floor to resist a uniformly distributed load. The bridging also stiffens the joists, and prevents them from turning sideways It is customary to insert rows of cross-bridging at from every five to eight feet in the length of the beams; and to be effective they should be in straight lines along the floor,

so that each strut may abut directly opposite those adjacent to it. The method of bridging shown in Fig. 1, and known as "cross-bridging," is considered to be by far the best, as it allows the thrust to act parallel to the axis of the strut, and not across the grain, as must be the case where single pieces of board are used.

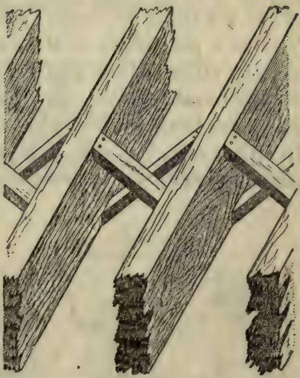


Fig. 5

The bridging should be of $1\frac{1}{4}$ inch by 3-inch stock, for joists $2'' \times 10''$ and under, and $2'' \times 3''$ stock for $12''$ and $14''$ joists.

Framing of Floor Beams.—In dwellings, tenement and lodging houses, it is a common practice to frame the ends of the tail beams into the headers, and very often the ends of the headers are framed into the trimmers. For light floors, with moderate spans, it is safe to frame the tail beams into a header, provided the latter is strong enough to carry the load and allow 1 inch in thickness for the mortising. Headers carrying not

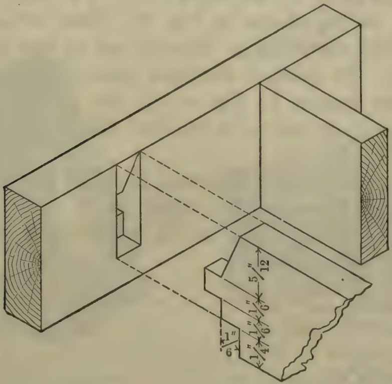


Fig. 6

more than two tail beams may also be framed into the trimmers, but all headers six feet long or over should be carried in

joist hangers or stirrups, and in warehouses and all first-class buildings all framing should be done by means of joist hangers.

As to the best shape and proportions for the tenon on the end of the tail beam or header, that shown by Fig. 6 gives probably as large a proportion of the strength of the timbers as it is possible to utilize, although for tail beams the author believes that a single tenon like that shown in Fig. 7 is fully as strong, especially when the header is built up of two-inch plank spiked together. In either case, if the floor is loaded to its full strength, the tail beam will split at the bottom of the tenon as shown in Fig. 8.

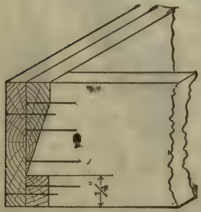


Fig. 7

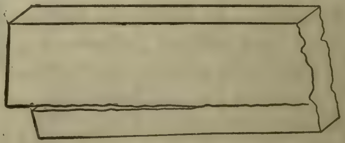


Fig. 8

Stirrups and Joist Hangers.—The first device used for framing headers to trimmers without mortising was the wrought-iron stirrup shown in Fig. 9. These are made either single or double, according to whether one or two beams are to be supported. To prevent the floor from spreading and thus permitting the header to slip out of the stirrup a joint bolt may be inserted, as shown in the two right-hand illustrations of Fig. 9.

To figure the strength of a stirrup, multiply the sectional area of the iron in square inches by 12,000 lbs.

The following sizes of iron should in general be used for the size of joist to be supported:

Size of Joist or Timber to be supported.	Section of Stirrup.
2×8 to 3×10	$\frac{1}{4}'' \times 2\frac{1}{2}''$
4×10 to 4×12	$\frac{3}{8}'' \times 2\frac{1}{2}''$
6×12 to 3×14	$\frac{3}{8}'' \times 3''$
8×12 to 4×14	$\frac{1}{2}'' \times 3\frac{1}{2}''$
6×14	$\frac{1}{2}'' \times 4''$
8×14 to 10×14	$\frac{5}{8}'' \times 4''$

Joist Hangers.—Aside from the matter of strength there are objections to the use of stirrups, in that if the timber on which they rest is not perfectly dry, the stirrup will settle by an amount equal to the shrinkage of the beam on which it rests,

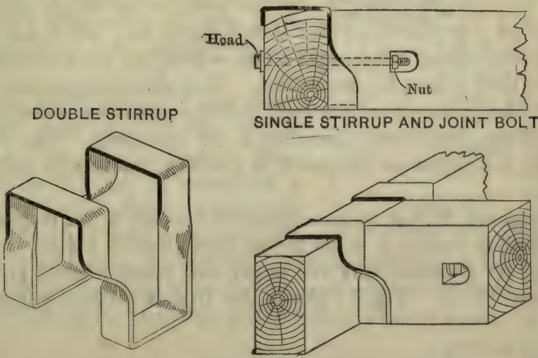


Fig. 9

and let the header down with it; the projection of the iron above the top of the timbers necessitates cutting out the flooring, and where the stirrups are exposed they do not present a neat appearance.

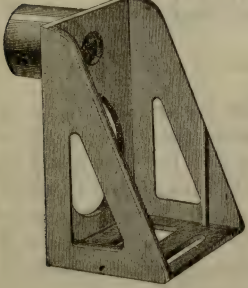


Fig. 10
Duplex Joist Hanger.

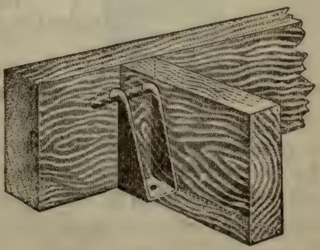


Fig. 11
Goetz Joist Hanger.

Within the past fifteen years several patented hangers have been placed upon the market, which are claimed to be superior

to the wrought-iron stirrup. The first of these in point of time was the Duplex hanger, shown in Fig. 10. This was quickly followed by the Goetz hanger, shown in Fig. 11. Both styles have been extensively used, and have proven perfectly satisfactory. Both are made in sizes to fit all regular sizes of joists or timbers, and have ample strength for the purpose for which they are intended. As shown by the illustrations, they are made to be inserted in round holes bored in the side of the carrying timbers, at or a little above the centre line. With these hangers the effect of shrinkage is reduced one-half, and the other two objections to the stirrup, previously mentioned, are overcome. The duplex hanger has ridges on the inside of the side brackets to hold the beam.

When the timber to be supported exceeds 6 ins. in breadth, the Duplex hanger is made in two parts, and is bolted to both beams, an illustration of the larger size hangers being given in Chapter XXII. Fig. 12 shows the Duplex I-beam hanger for

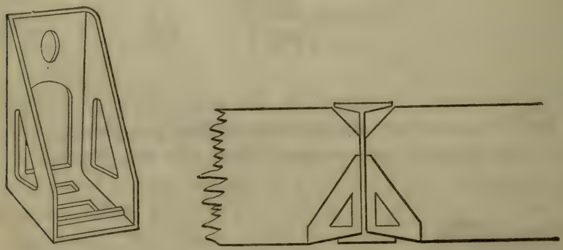


Fig. 12
Duplex I-Beam Hangers.

framing floor joists to I-beams. These hangers are made to exactly fit into the flange of the I-beam, they have a rib in bottom of hanger $\frac{3}{8}''$ high, which serves as a tie when the joist is placed in the hanger, and they provide a bearing of $4\frac{1}{2}$ inches for the joists. These hangers are made to carry all regular sizes of joists from $2'' \times 6''$ up to $6'' \times 16''$, and in the opinion of the author offer the best device for framing wooden joists to I-beams of the same depth. The hangers are all of uniform height and a $\frac{3}{4}''$ hole punched $6''$ from the bottom of the beam will fit any of them. The hangers are bolted to the web of the I-beam.

Fig. 13 shows a similar hanger made to support the wall end

of a floor joist. The writer believes this to be much superior to the method of building the joist into the wall, as it absolutely prevents dry-rot, and permits the joist to fall, in case of fire, without throwing the wall. It also gives the weight a good bearing on the wall.

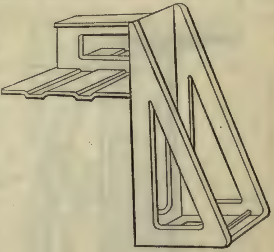


Fig. 13
Duplex Brick Wall Hanger.

Other illustrations of wall hangers are given in Chapter XXII.

The Van Dorn Hanger, illustrated by Fig. 14, is essentially a stirrup forged from high-grade steel. The few tests that have been made would seem to indicate that it possesses a little more resistance to bending than the ordinary stirrup, while it gives a wider bearing for the joist, and presents a much neater appearance.

Fig. 15 shows the same hanger riveted to a bent iron plate, to build into brick walls.

When the hanger is to be used over a steel beam the upper ends are bent to fit over the flange of the beam, as in Fig. 16.

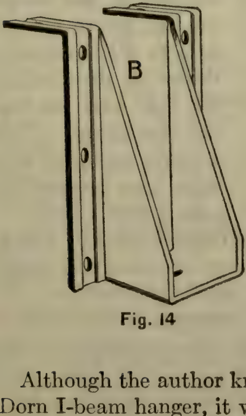


Fig. 14

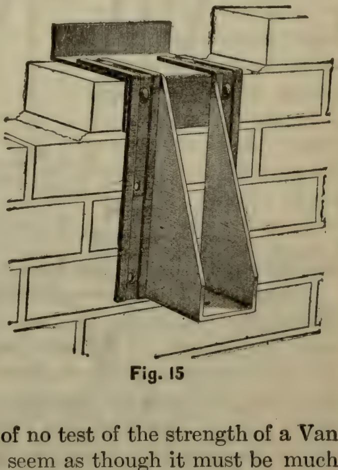


Fig. 15

Although the author knows of no test of the strength of a Van Dorn I-beam hanger, it would seem as though it must be much stronger than the pattern made for wooden beams, on account

of the clinch over the flange of the I-beam. **The Van Dorn** hangers have been used in many important buildings.

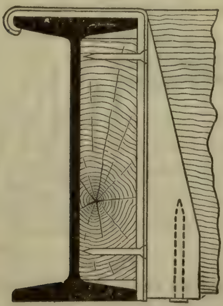


Fig. 16

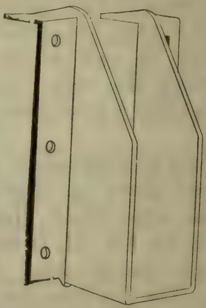


Fig. 17
National Hanger.

Figs. 17 and 18 show two other patented joist hangers of the stirrup type, which are forged from plate steel. Both of these hangers are also made for building into brick walls, and to go over steel beams. The national hanger is a particularly good one on account of the flange on top, which should help to a considerable degree in distributing the load over the top of the beam. The larger hangers of this style have holes in the top for large spikes. The Lane hanger is made very light.

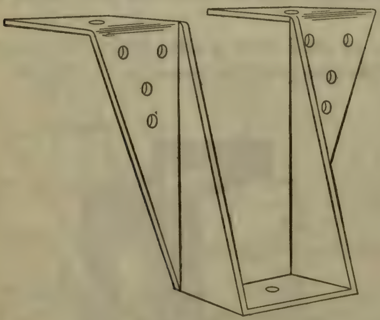


Fig. 18
Lane Hanger.

Comparative Strength of Different Styles of Joist Hangers.—Although the tests that have been made to determine the strength of different hangers are few in number, still enough have been made to show that any one of the hangers described, including the common stirrup, are abundantly strong for any *single floor beam* not exceeding 4″×14″ in size. It is only in the case of a header or trimmer which supports a considerable floor area that the strength need be considered at all.

From tests made at the Massachusetts Institute of Technology, and later at St. Louis, it would appear that the Duplex hangers are affected the least of any under extreme loads. A two-part hanger, carrying a 10×14 inch girder, sustained a load of 38,000 lbs. without injury to the hanger itself. A similar hanger held until loaded to 39,550 lbs., when one side broke off short under the nipple projecting into the timber, the condition of the hanger after failure being shown by Fig. 19.

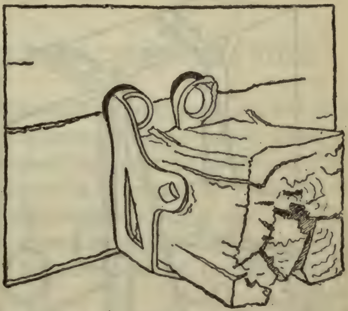


Fig. 19

A common stirrup made from $\frac{3}{8}'' \times 2\frac{1}{2}''$ wrought iron failed under a load of 13,750 lbs. by bending and pulling over the header, as shown in Fig. 20. A $6'' \times 12''$ Van Dorn hanger "began to straighten out under a load of 13,300 lbs., and failed as in Fig. 21 at a load of 18,750 lbs." *

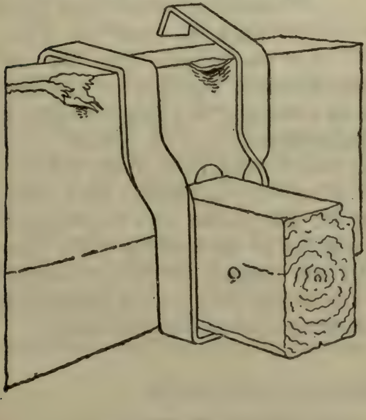


Fig. 20

Single hangers of the stirrup type *do not break*, but fail by the bending up of the portion which lays over the top of the header, as in Figs. 20 and 21. They also appear to crush the wood under them, particularly at the edge, to a very much greater degree than does the spool of the Duplex hanger.

With a *double* stirrup the ultimate strength is measured by the strength of the iron. Thus a double stirrup made of $\frac{3}{8}'' \times 2\frac{1}{2}''$ wrought iron was loaded up to 57,650 lbs. (28,825 lbs. on each side), when it broke

* Chas. E. Fuller, M.E.D., Dept. M.I.T.

at one of the lower corners. A single stirrup would of course be just as strong if it could be kept from bending. In actual construction the flooring over the beams would to some degree prevent the top of a stirrup from springing up. The tests that have been made of the Duplex hangers show conclusively that where only a single hanger is used the holes which are bored in the header do not affect its strength, at least when the load is within the safe limit, and a test made at Baltimore, Md., Aug. 24, 1904, with $2'' \times 12''$ joists, spaced 12 ins. on centres and suspended by duplex hangers let into a header formed of three $3'' \times 12''$ joists, spiked together, would seem to prove that even when the holes are 12 ins. on centres they do not weaken the header. The only record of the failure of any form of hanger when in actual use in a building, of which the author is aware, is that of a case in Minneapolis, where a portion of six floors of a warehouse fell, on Nov. 7, 1902, through the failure of a wall hanger made from a $4'' \times 2'' \times \frac{1}{4}''$ structural steel angle, sheared and bent as in Fig. 17, and riveted to a bearing-plate $8'' \times 16'' \times \frac{1}{4}''$. The failure was due to the crushing of the outer edge of the brickwork under the hanger, and the consequent bending up of the top. The actual load on the hanger was about 15,000 lbs. See *Engineering News* of Nov. 20, 1902.

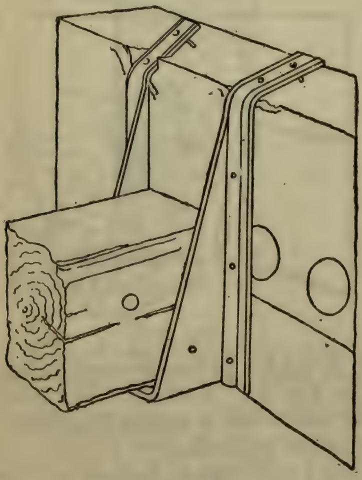


Fig. 21

CHAPTER XXII.

MILL AND WAREHOUSE CONSTRUCTION.

Mill Construction.—This term is commonly used to designate a method of construction brought about largely through the influence of the factory mutual insurance companies of New England, and especially through the efforts of Mr. William B. Whiting, whose mechanical judgment, experience, and skill as a manufacturer were for many years devoted to the interests of these companies and to the improvement of factories of all kinds.

The extended use of this system, and the improvements that have been made in it during the past twenty years, is probably due more to the influence of Mr. Edward Atkinson, president of the Boston Manufacturers Mutual Insurance Co. and director of the Insurance Engineering Experiment Station at Boston, than to that of any other individual.

The motive of mill construction is to reduce the fire risk to its lowest point, without going to the expense of fire-proof construction. The mill construction recommended by the Factory Mutual Companies has proved to be so safe as a whole, and such factories have been covered by mutual insurance at so little cost as to render it wholly inexpedient, or even unnecessary, for the owners of textile factories and workshops to take any other method into consideration.

The entire subject of Slow-burning or Mill Construction, as applied to factories, is most admirably described and illustrated in Report No. 5, of the Insurance Engineering Station, No. 31 Milk St., Boston, Mass.,* from which the author has by permission taken the following illustrations and descriptions.

What Mill Construction Is.

[From Report No. 5 of the Insurance Engineering Station.]

1. Mill construction consists in so disposing the timber and plank in heavy solid masses as to expose the least number of

* This Report may be procured for 25 cents.

corners or ignitable projections to fire, to the end also that when fire occurs it may be most readily reached by water from sprinklers or hose.

2. It consists in separating every floor from every other floor by incombustible stops,—by automatic hatchways, by encasing stairways either in brick or other incombustible partitions,—so that a fire shall be retarded in passing from floor to floor to the utmost that is consistent with the use of wood or any material in construction that is not absolutely fire-proof.

3. It consists in guarding the ceilings over all specially hazardous stock or processes with fire-retardent material such as plastering laid on wire lath or expanded metal, or upon wooden dovetailed lath, following the lines of the ceiling and of the timbers without any interspaces between the plastering and the wood; or else in protecting ceilings over hazardous places with asbestos air-cell board, sheet metal, Sackett wall board, or other fire-retardent.

4. It consists not only in so constructing the mill, workshop, or warehouse that fire shall pass as slowly as possible from one part of the building to another, but also in providing all suitable safeguards against fire.

What Mill Construction is Not.

1. Mill construction does *not* consist in disposing a given quantity of materials so that the whole interior of a building becomes a series of wooden cells, being pervaded with concealed spaces, either directly connected each with the other or by cracks through which fire may freely pass where it cannot be reached by water.

2. It does *not* consist in an open-timber construction of floors and roof resembling mill construction, but of light and insufficient size in timbers and thin planks, without fire-stops or fire-guards from floor to floor.

3. It does *not* consist in connecting floor with floor by combústible wooden stairways encased in wood less than two inches thick.

4. It does *not* consist in putting in very numerous divisions or partitions of light wood.

5. It does *not* consist in sheathing brick walls with wood, especially when the wood is set off from the wall by furring, even if there are stops behind the furring.

6. It does *not* consist in permitting the use of varnish upon woodwork over which a fire will pass rapidly.

7. It does *not* consist in leaving windows exposed to adjacent buildings unguarded by fire-shutters or wired glass.

8. It is dangerous to paint, varnish, fill or encase heavy timbers and thick plank as they are customarily delivered, lest what is called dry-rot should be caused for lack of ventilation or opportunity to season.

9. It does *not* consist in leaving even the best-constructed building in which dangerous occupations are followed without automatic sprinklers, and without a complete and adequate equipment of pumps, pipes, and hydrants.

10. It does *not* consist in using any more wood in finishing the building after the floors and roof are laid than is absolutely necessary, there being now many safe methods available at low cost for finishing walls and constructing partitions with slow-burning or incombustible material.

It follows that if plastering is to be put upon a ceiling following the line of the underside of the floor and the timber, it should be plain lime-mortar plastering, which is sufficiently porous to permit seasoning. The addition of the skim coat of lime putty is hazardous, especially if the top floor is laid upon resin-sized or asphalt paper. This rule applies to almost all timber as now delivered.

All the faults above recited have been committed in buildings purporting to be of mill construction, and all form a part of the common practice in "combustible architecture."

Standard Mill Construction.

Fig. 1 shows a partial cross-section through a mill of the customary or standard type as revised to Nov., 1902.

If additional stories are required the walls may be increased in thickness according to the number of stories added, after a computation of the loads which a standard factory may be called upon to sustain.

Fig. 2 shows an enlarged view of the exterior of two bays, with recessed panels between the piers. Fig. 3 shows the common form of cast-iron pintle, which serves as a cap for the lower column, and a support for the upper column.

These illustrations are only intended to give general directions for slow-burning or mill construction. They should always be

adapted to the special conditions of each site and of each art for which the buildings are used.

When a span exceeds twenty-two feet it is judicious to add to the support by hackmatack or iron knees projecting from wall and posts. These knees or braces are not deemed necessary even on spans of twenty-five feet when the timbers are of ample dimension. They have sometimes been put into old mills of high and narrow type and have stopped serious vibration in the upper stories. If used, they must be kept bolted closely to timbers and posts, and care should be taken

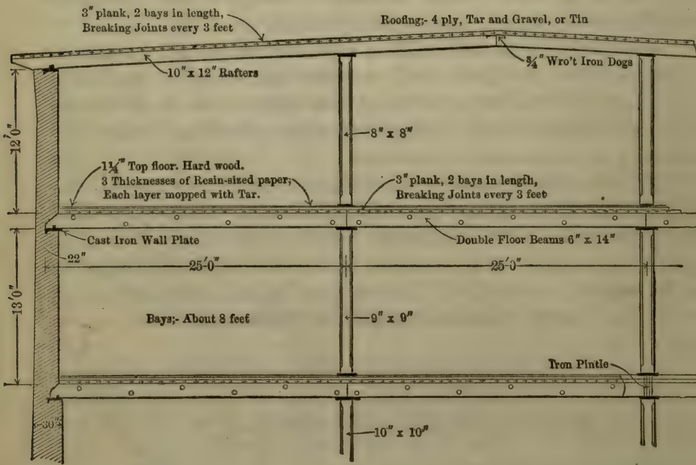


Fig. I
Standard Construction.

that the load on each side should be practically the same. They are necessary in the self-sustaining frame (Figs. 9 and 11). In computing the size of the timbers in ratio to the working load, regard must be given not only to the weight which is to be carried, but also to the character of the mechanism which is to be operated upon the floor. Beams of sufficient strength to support the weight may be caused to vibrate or deflect under the action of the machinery; therefore the two factors of weight and vibration must be considered in determining the size or depth of the beams that may be made use of.

"We do not approve what has sometimes been miscalled mill

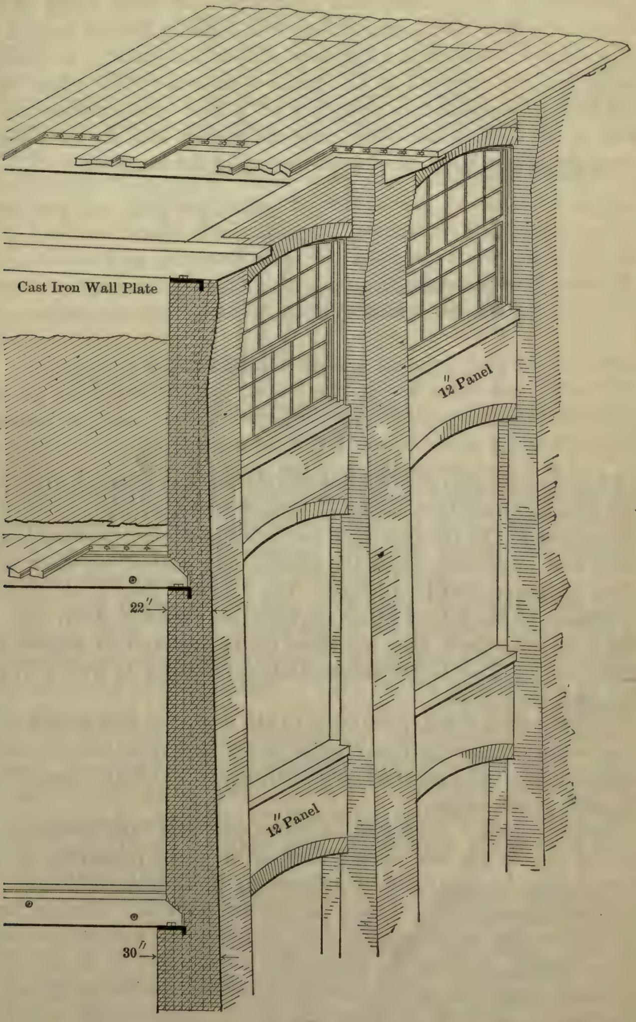


Fig. 2
Detail of Side Walls.

construction, i.e., longitudinal girders resting upon posts and supporting floor-beams spaced four feet, more or less, on centres. This mode of construction not only adds to the quantity of wood used, but the disposal of the timbers obstructs the action of sprinklers, prevents the sweeping of a hose stream from one side of the mill to the other, and the girders also obstruct the most important light, that from the top of the windows."

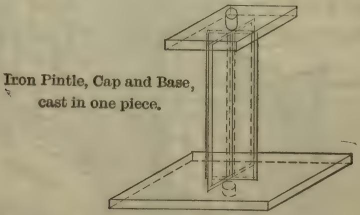


Fig. 3

The standard plan calls for but one thickness of boards laid over the planks, with three layers of resin-sized paper mopped with tar between. In the best mills lately built a board flooring has been laid diagonally upon the plank, over that a top floor of birch or maple, laid lengthwise. The diagonal floor gives great resistance to the lateral strain or vibration; the top floor, especially in alleyways, can be more easily repaired or replaced when worn. This intermediate diagonal flooring is well worth the additional cost.

The following notes, pertaining to the details of mill construction, are the result of many years' study and observation, and should be carefully noted when preparing plans and specifications for mill construction.

Timbers, unless known to be absolutely and fully seasoned, should not be encased in any kind of air-proof plastering, nor should they be painted with oil-paints; whitewash, kalsomine, and water-paints may be used as they are porous. Timbers or plank may also be covered in with common lime mortar laid on wire lathing, provided no skim coat of lime putty is added. Ordinary plastering unskimmed is sufficiently porous to permit seasoning. As a rule timbers may be left unprotected, except in very dangerous places, since any fire which will seriously impair and destroy a heavy timber will already have done its work upon other parts of the structure.

In many instances it may be preferable to substitute compound beams for single timbers, made by securing two or more beams or thick planks side by side; it being often easier to obtain well-seasoned lumber in smaller dimensions; such compound beams, of which the parts may be slightly separated by spaces for ventilation when put together, are less subject to decay.

Weaving mills can be made more rigid and more capable of resisting the vibration caused by the motion of looms by laying the top floor across the plank and parallel to the beams, nails being driven in diagonal rows. This may brace the floor as firmly as diagonal boarding, and it avoids the increased expense in construction and repairs which ensues from the adoption of that method.

The edges of the floor plank should be kept clear of the faces of the brick walls by about half an inch, in order to obviate the danger of cracking the walls, which sometimes occurs from the swelling of the plank when laid close against them. These cracks must be covered by strips or battens both above and below.

To protect the contents of floors below, three thicknesses of tarred paper should be placed between the floor plank and top floor, each layer to be mopped with tar, asphalt, or similar material, care being taken to break all joints.

Basement floors can be laid solid upon the natural soil if it is dry, or upon rock or cinder filling, by covering either with a suitable layer of *coal-tar* concrete. Upon this concrete place an underfloor of two-inch plank. Then lay the top flooring across the plank, and nail in the usual manner. Sills under the plank are not thought to be necessary to the preservation of the floor. If extra support is required to sustain machinery more firmly than it can be upon a plank and board floor, independent foundations of masonry are generally preferred. *Cement concretes* may absorb moisture and promote the decay of timber or plank laid upon them.

In view of the difficulties which have frequently occurred in preserving basement floors of the ordinary timber construction for lack of suitable ventilation underneath, and also in view of the rapid decay of timber and plank floors in bleacheries, dye-works, print-works, and the like, where they quickly become saturated with moisture, artificial stone floors are being laid in many of the modern plants.

If the mill is to be heated by conveying steam through pipes,

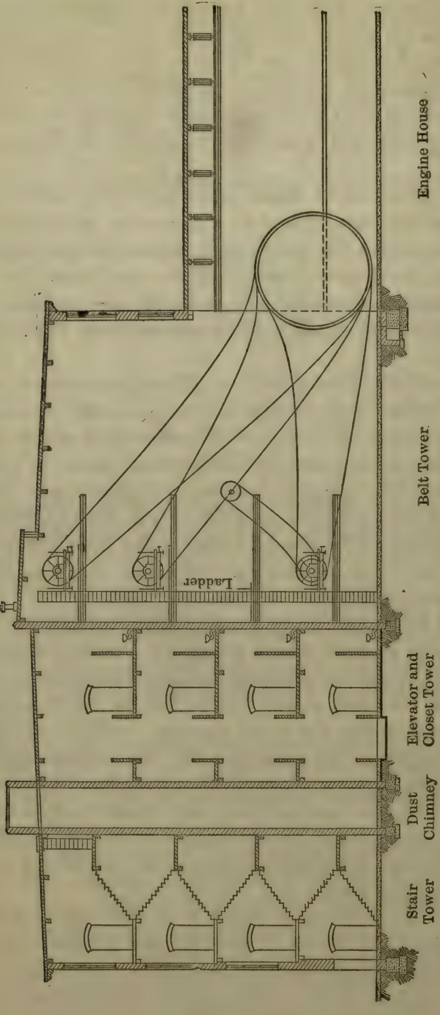


Fig. 4
Section through Belt, Stair, and Elevator Towers.

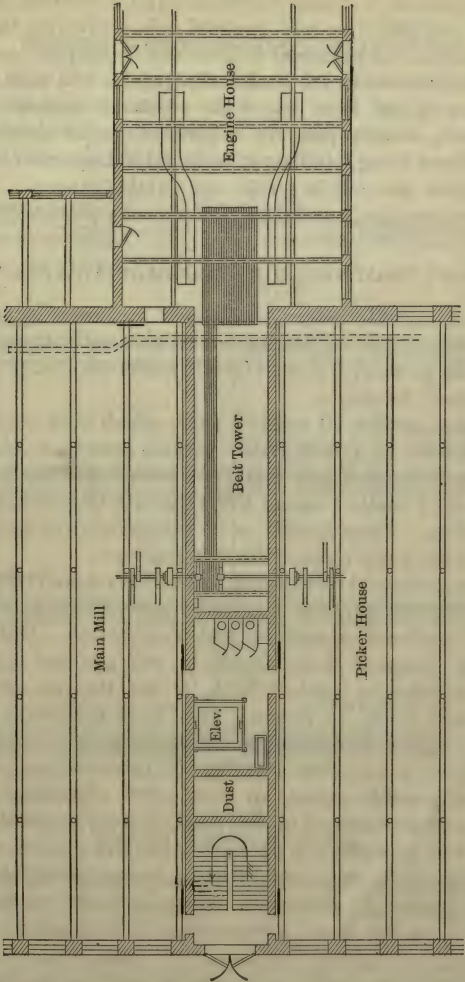


Fig. 5
Showing Location of Stairs, Elevator, Belt Tower, etc.

such pipes should be hung overhead. If the modern method, which is probably the best method, of conveying the heat through ducts in the plastered walls should be adopted, provision will be made thereto in the construction of the mill wall.

The carrying off from the walls of about one-half a roof corresponding to this plan, in a hurricane, calls attention to the necessity of tying, binding, or bolting the timbers of the roof to the walls of the mill in a safe and suitable manner. This is the common practice, but the necessity is sometimes overlooked.

Belt, Stairway, and Elevator Towers.

Stairways should always be located in towers or sections of the building, cut off by incombustible walls from all the rooms of the factory, the entrances to each room being guarded with standard fire-doors.

In modern practice all belts or ropes which may be used for the transmission of power to the various rooms are placed in incombustible vertical belt chambers, from which the power is transmitted by shafts through the walls into the several rooms of the factory. There should be no unprotected or unguarded openings in the inner walls of this belt chamber.

Elevator shafts and belt towers or chambers should be guarded by fire-doors and covered overhead by skylights, glazed with thin glass, protected underneath with wire netting. Hatchways outside the fire-proof shafts should be well guarded by automatic or self-closing hatches, both to stop the passage of fire and to assure safety to persons. The most important feature in what is called slow-burning construction is to make each and every floor continuous, avoiding belt holes and open ways to the utmost possible extent, so that a fire originating in any one room may be confined to that room or story, if possible.

Figs. 4 and 5 illustrate a partial section and plan of a cotton mill, showing belt, stair, and elevator towers arranged on the above principle. It should be noted that the water-closets are located in the tower rather than in the manufacturing rooms.

The boiler-house should be located beyond the engine-room, and separated from the latter by a brick wall with doorway protected by a standard automatic fire-door.

Standard Storehouse Construction.

Figs. 6, 7, and 8 represent salient points in design for a mill storehouse several stories in height, and include many features found useful in practice for convenience in operation and also securing the greatest measure of resistance to fire.

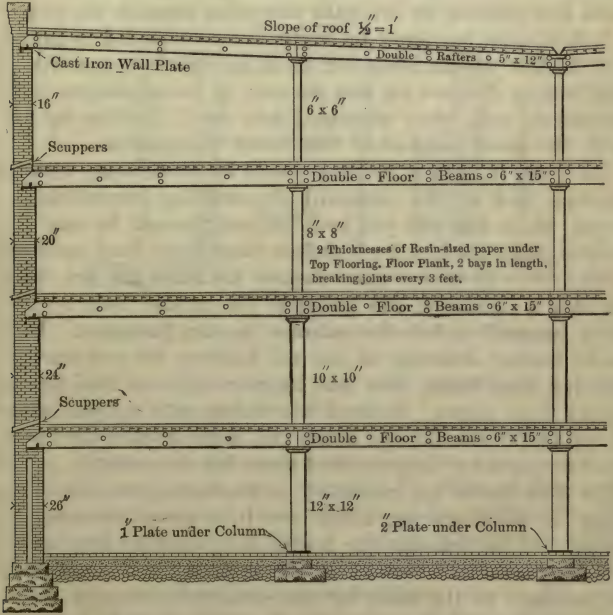


Fig. 6

One-half of Transverse Section.

The size of columns and beams is only for example, differing according to load and span, the drawings not being intended to take the place of the services of any mill engineer, but rather to assist in such work. It is important that the floor beams should be designed to sustain the greatest load ever to be placed on them, and the stories should be made low enough to prevent overloading, and also to prevent bales of material from being piled to great height, the preferable method being to place bales on end.

These floors, with beams of 20 feet span, laid 8 feet on centres, will sustain a load of 180 pounds per square foot, which is as much as would be required for raw material or finished goods of a textile or paper mill. The heavy drugs and dyestuffs would be placed on the ground floor.

For convenience, as well as to separate the different hazards of raw material and finished goods, the building may be divided into sections by fire-walls extending through the roof.

A storehouse one story in height is recommended in preference to this design whenever there is sufficient quantity of level land at disposal for this purpose, as being cheaper, more convenient, and, when separated into small divisions by fire-walls, the safest method of storehouse construction.

The floors in such a building should be continuous, without openings, and of the standard slow-burning construction—a type which has not yet been burned through by any fire starting under such a floor, unless there have been openings in the floor. To reduce water damage the floors are not level, but have a camber of two inches in the middle made by iron plates inserted under the columns in the basement. If it should become desirable to use the building for any purpose requiring level floors, they can be reduced to a level by removing these plates. Inclined iron tubes, with a light swinging cap on the outside, laid in the wall at the level of the floors, act as scuppers for the purpose of removing any water.

The floor-beams are preferably of Southern pine bolted together in pairs, leaving about one inch space between the beams. At the columns the beams are joined by dogs made of three-fourths inch round iron, driven in at the top, and they are anchored to the walls by cast-iron wall-plates, to which they are secured by means of a rib which fits into a groove crossing the underside of the beam. It is important that there should be a small space at each side and at the end of the beam, in order to allow free ventilation, for the purpose of preventing dry-rot. The Goetz box-anchor is a special form of wall-plate which is especially adapted to such purposes.

The underfloor is made of spruce plank, generally three inches thick, planed on the underside, and grooved at the edges, and fitted with hardwood splines. These plank are two bays in length, breaking joints at least every three feet.

The floor is smoother if laid across the line of plank, and the travelling loads moved in or out of the storehouse are

better distributed than when the top floor is laid parallel to the plank. To protect the contents of floors below, three thicknesses of tarred paper should be placed between the floor plank and top floor, each layer to be mopped with tar, asphalt, or similar material, care being taken to break all joints. The floor should not be secured to the walls, but a narrow strip

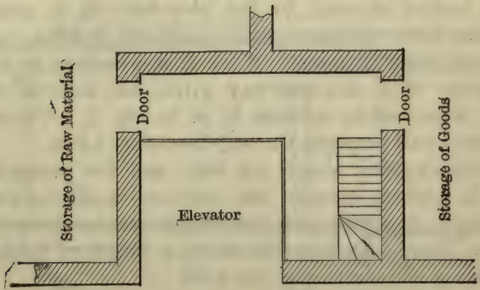


Fig. 7
Showing Stairway Tower Inside Walls of Building.

laid around the edges of the floor and fastened to the wall covers any openings due to shrinkage.

Floors of storehouses may have a slight pitch toward the centre, draining in the same method as the roof is drained; or if the other type of roof is adopted, the floors of storehouses may have a slight pitch from centre to walls, draining through scupper-holes indicated in Fig. 6. Goods raised on low skids may then be very free of water damage.

The columns should be square Southern pine or oak, with iron cap, pintle, and base, preferably cast in one piece, and secured to the under side of the beam by six-inch lag screws. The caps should be large enough to give the beams ample bearing surface.

In a warm storehouse it is preferable that the roof should slope towards the centre one-half inch to the foot, and that the gutter should slope towards the drain-pipe one-twentieth of an inch to the foot. In a cold storehouse it is considered necessary that the roof should slope one-half of an inch towards the walls as in Fig. 1, and if there is a gutter, the conductors should be carried outside of the building.

Access to the various stories is obtained by means of a tower

outside the main building and extending above the roof, containing stairways, elevator, and water-pipes. At each story of the tower open galleries communicate to the rooms on that level. A doorway from the upper story of the tower affords a ready means of reaching the roof. It is often a matter of great convenience if the doorway at the first story of the tower is made large enough, and at the outside grade, so that a wagon can be backed directly to the elevator. It is unnecessary to provide these elevators with automatic hatches, as guard-gates serve every purpose. When it is necessary to construct the elevator wells and stairway within the lines of the main building it is best to arrange it as in Fig. 7. The omission of the outer wall prevents the way becoming a flue.

The walls extend above the roof, and the parapet should be laid in cement, because the moisture readily absorbed by brick would otherwise pass downward and render walls in the top story damp. In some instances a course of brick dipped in coal-tar is laid above the roof level.

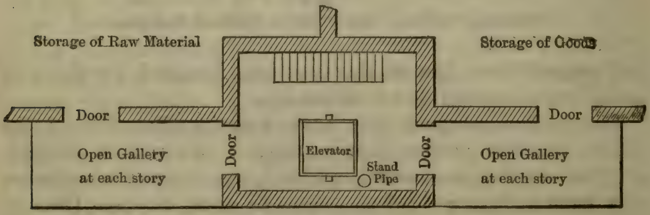


Fig. 8
Showing Stairway Tower at Side of Storehouse.

In addition to yard hydrants near the buildings there should be a six-inch standpipe in the tower, the supply to which should be controlled by an out-of-door Post indicator valve at a safe distance from the building, with two $2\frac{1}{2}''$ hydrants and hose at each story, and at the top story of the tower, the standpipe branches to a Morse or Phillips monitor nozzle on the roof, if there are any adjacent buildings which might be reached by streams from this position. A set of plugs for the roof drain-pipes will allow the roof to be covered with water in case the property is endangered by sparks from burning buildings.

Storehouses should be provided with automatic sprinklers.

When so protected the water can be shut off in the winter; or if an air system be adopted it may be applied only in the winter; water being kept directly upon the sprinklers in warm weather.

Mill Construction with Self-sustaining Frame.

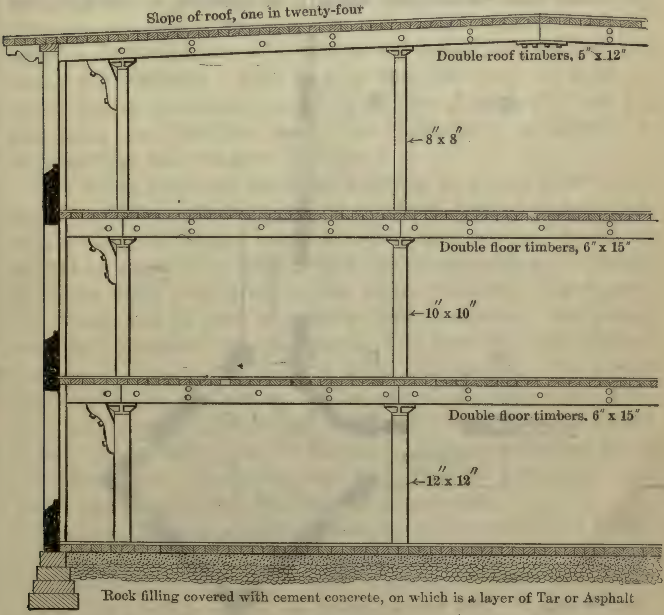


Fig. 9
Partial Transverse Section, Showing Self-sustaining Frame.

Figs. 9, 10, and 11 show suggestions for mill construction in which the interior framework is self-sustaining and independent of the walls, except that the outer posts serve to brace the walls. Regarding this construction the Boston Manufacturers Mutual Fire Insurance Co. says:

"It is suggested that the framework of a factory itself should consist of two parts, *first*, of two outer lines of posts passing around the whole building, joined and braced together either with hackmatack knees, angle irons, or iron cross-ties; this

outer framework carrying the alleyways to be so constructed that it may be put up separately from the outer walls and also separately from the inner posts and floors on which the machinery is to be placed, so that this part of the frame bearing the outer alleyways will stand alone.

"*Second*, The interior framework and the floors upon which machinery is to rest may be adjusted and connected with the outer lines of support already designated, so that in the

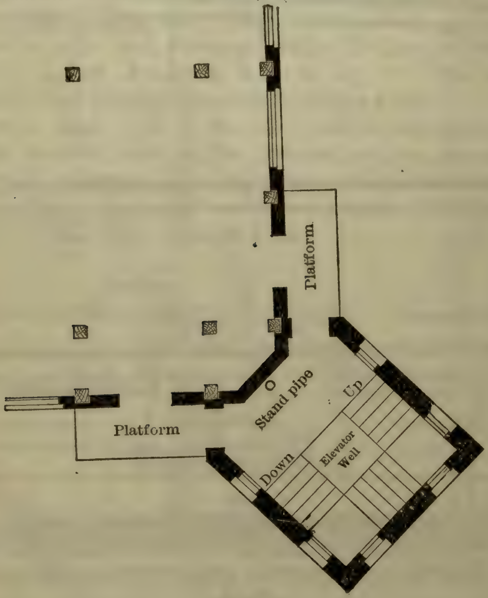


Fig. 10
Partial Plan, Showing Locations of Posts and Staircase Tower.

event of the burning of any timber, the giving away of any post, or accident to any part of the floor, any section of machine-bearing floor may fall out without bringing a severe strain upon the outer line of posts or upon other parts of the frame of the mill.

"The self-sustaining frame being established and covered in, the question may follow rather than precede, as to what the material of the outer walls should consist of. In very many arts it is important that the outer walls should consist in large

measure of glass, especially since the adoption of double-glazed windows and the suggestion of ribbed glass for glazing has solved the problem of the transmission of light, while the double glazing in the same sash obstructs the passage of heat and cold in a very effectual way, thus avoiding the condensation of the moisture within.

"These windows are to be carried to the under side of the roof, against the plate, and to the under side of each floor, close to the ceiling, therefore the only remaining part of the outer wall to be dealt with will be the space between and beneath the windows. This part of the wall may be built of brick, but as no dependence is to be placed upon the wall for sustaining the building itself, the walls may be lighter or thinner than the common practice.

"It being premised that this building is planned for a detached position, where it will be free from hazard of fire from neighboring buildings, large windows may be put in which cannot be guarded by fire-shutters, the consideration of danger from fire being only given to the interior hazard. Upon such conditions and in such a position other materials than brick may be considered for the walls. For instance, the timbers may be so disposed as to show upon the outside. The entire framework for the window to be placed between these timbers may then be constructed so that it can be brought to the factory ready to be put into its place, all window-frames being interchangeable. This structure may then be protected both inside and outside with incombustible materials.

"Where there is an outside hazard the windows must be diminished in width, with an equal or greater area of wall surface between, in or upon which automatic shutters may be placed for closing up the windows against fire. In such positions the outer wall can only safely consist of brick of such thickness as may be suitable to each case."*

Fig. 11 shows a detail of outside wall consisting principally of windows, the frames filling the entire width between the posts. The outside of the posts and the spaces under the windows may be covered with plank, and then with metal or sheathing lath and rough plaster; the entire wall being carried by the posts. In this mill there is but a single line of outer posts, i.e., no alley.

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* Hollow concrete walls should be even better than brick.—Author.

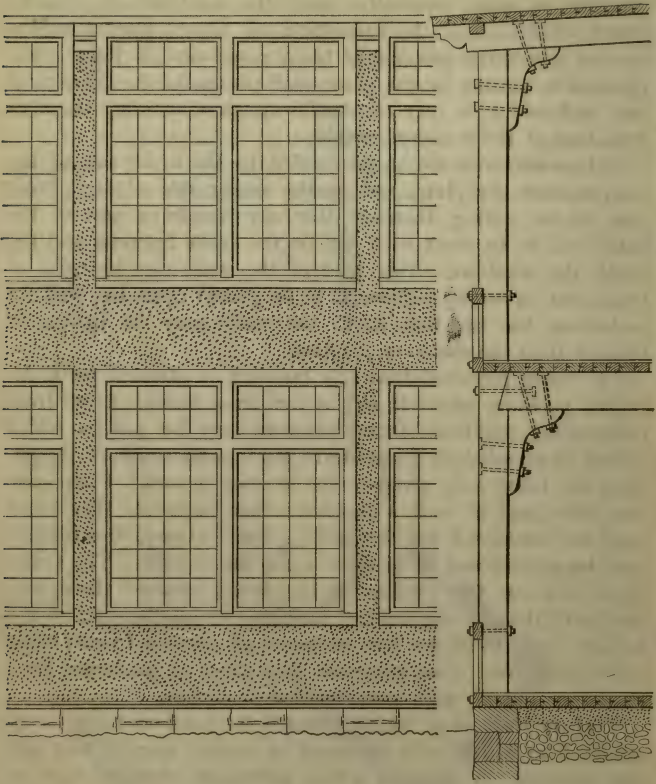

Fig. 11
Outside Walls of Wood and Plaster Supported by the Frame.

Example of One-story Slow-burning Machine-shop,

"COVERING IMPROVEMENTS WHICH HAVE BEEN DEVELOPED BY EXPERIENCE IN THE CONSTRUCTION AND USE OF ONE-STORY FACTORY BUILDINGS UP TO THE PRESENT DATE." *

For workshops on cheap, level land, especially where the stock is heavy, one-story buildings have proved to be more

* Edward Atkinson, President. November, 1902.

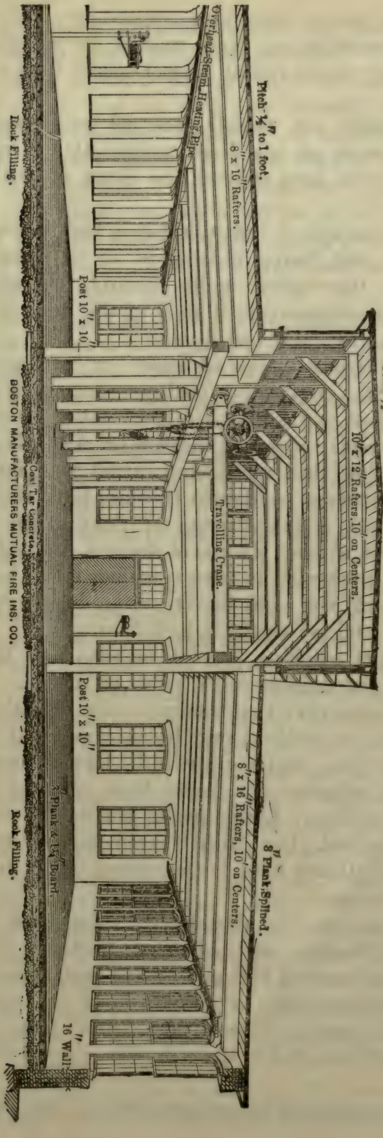

Fig. 12
Section.

economical in cost of floor area, supervision, moving stock in process of manufacture, and repairs to machinery—many kinds of which can be run at greater speeds than when in high buildings.

Such buildings are readily warmed and ventilated, and heavy plank roofs are free from condensation in cold weather; the large window area reduces the hours of artificial illumination.

Forced circulation of heated air is a very desirable method of heating a mill, being economical as to maintenance and repairs, and thoroughly under control. Overhead steam-pipes are very satisfactory, if used in the ratio of one foot of 1¼-inch pipe to 70 cubic feet of air.

Floors.— Floors over an air space or on cement are subject to dry-rot. Asphalt or coal-tar concrete is softened by oil, and the dust will wear machinery, unless covered by flooring. Floors made by laying sleepers on six inches of pebbles, tarred when hot, then two inches tarred sand flush to top of sleepers and covered by double flooring, have remained sound since 1865; but double flooring at right angles can be laid on the concrete without the use of sleepers, and nailed together. It is usually preferable to secure nailing strips to stakes four feet apart each way and driven to grade, concrete flush to top of strips, and lay single 1½-inch flooring.

Walls.—Piers reaching to roof timbers, and light walls to window-sills are finished with slope on inside. To increase the window area over that shown in the elevation, the brick piers between the windows may be narrowed and made thicker so as to give the requisite strength, leaving more space for light. Large windows are placed high, and the sashes separated by a mullion. Lower sashes should be stationary and glazed with ribbed glass, with transom sash or window ventilators above. If the light is too strong, apply to glass white zinc and turpentine.

Monitors may well be glazed with ribbed glass.

Columns.—Wooden mill columns, Southern pine or oak, safely sustain loads of 600 pounds per square inch; a square column is stronger than a round one of the same diameter. They should have a 1½-inch core bored from end to end, and two half-inch holes through the column near to each end.

The columns should be securely held at each end, the base

resting on iron plates projecting above the floor level, and the caps at the top bolted to the roof beams.

Roofs.—Double or solid timbers of Southern pine support the roof plank, and the ends pass through the wall, and are finished as brackets to the cornice; or another plan often adopted is to make a projecting brick cornice covering the ends of the roof timbers, thus avoiding the exposure to an outside fire. The beams are anchored to plates in the walls by means of tongues which project into grooves across the lower side of the beams. Beams should not be painted or varnished until thoroughly seasoned.

The roof plank should be two bays in length, breaking joints every three feet. There is no need of gutters, but a concrete walk at the ground level, sloping toward drains, will take the water from the roof. Do not drive nails upward into the roof plank, as moisture will condense and drop from the heads.

Monitors must be of solid construction.

The roof should be tied by binding or bolting the timbers of the roof to the walls of the mill in a safe and suitable manner. This is the common practice, but the necessity is sometimes overlooked.

The saw-tooth roof is taking the place of the monitor in weaving and other buildings. We do not supply any plan for this type because it requires, in each case, the service of a competent mill engineer and constructor to plan the roof so as to meet special conditions, and to supervise the work.

Roofing Material.—Mill roofs are almost always flat, as shown in the foregoing specifications, and are most commonly covered with coal-tar, pitch, and gravel, although asphaltic compositions are often used and occasionally tin roofing is used. Cotton duck or canvas has been used for covering mill roofs to some extent, but does not appear to have proved very satisfactory, except on small buildings and for covering platforms, etc. Canvas roofing will stand harder usage than any of the materials above mentioned, as is shown by its continued use on the decks of vessels and steamers. Duck or canvas for roofing purposes should be what is termed "12 ounce," weighing 16 ounces to the square yard. It should be tightly stretched, and tacked with seventeen-ounce tinned carpet-tacks, the edges being lapped about an inch. If the roof planks are rough, or not of an even thickness, a layer of heavy roofing paper should be laid before the duck is put down.

After the duck is laid, it should be thoroughly wet, and then painted with white lead and boiled linseed-oil before it becomes dry; which makes it water proof. To protect from fire, give it two more coats of white lead, and over this a coat of iron-clad paint. Instead of the four coats of white lead and oil, the duck may be saturated with a hot application of pine-tar thinned with boiled linseed-oil. This has been found to work perfectly. The iron-clad paint should be applied, whichever method is used.

Partitions.—Partitions used for dividing a mill into sections should be built of brick, concrete or porous terra-cotta tiling. Where a room is to be partitioned off the partitions may be built of two-inch tongued and grooved plank set vertically (so as to form a solid partition), and plastered both sides, either on wire, or on dovetailed sheathing lath. Such partitions have been found to work well after a trial of twelve years, and offer effectual resistance to fire.

Two-inch solid partitions of plaster on metal lath wired to light iron studs may also be used.

Mill Doors and Shutters should be built of two thicknesses of inch boards covered on all sides with tin, as described in Chapter XXIII.

For information relating to appliances for the fire protection of mills, the reader is referred to the Insurance Engineering Station, Boston.

Patented Systems of Mill Construction.

Mr. Chas. A. M. Praray, Mill Engineer of Providence, R. I., has patented a system of mill construction which he has designated as the "Praray Improved Construction." The special points in which this system differs from the regular mill construction are, the construction of the walls, the shape of the post caps, and the supporting of the floors and roof, independent of the walls.

The walls are built as a continuous series of bay windows, with a hollow brick pier between, and with a supporting column in the centre of each bay. Fig. 13 shows a plan of one bay and of the column supporting the floor, while Fig. 14 shows a vertical section through the same. Fig. 15 shows a detail of the post and girder connection. The posts are cut down to a square where they pass through the girder

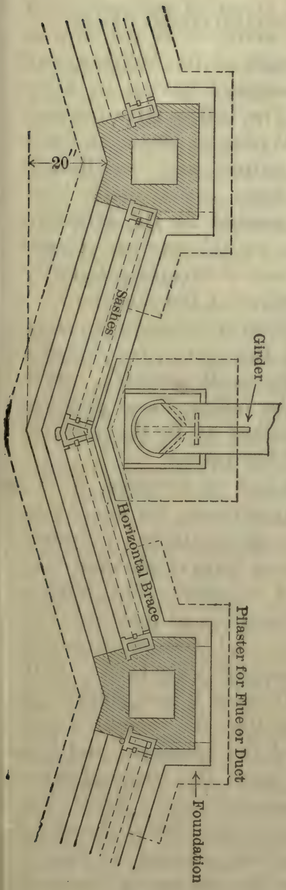

Fig. 13
Plan of One Bay.

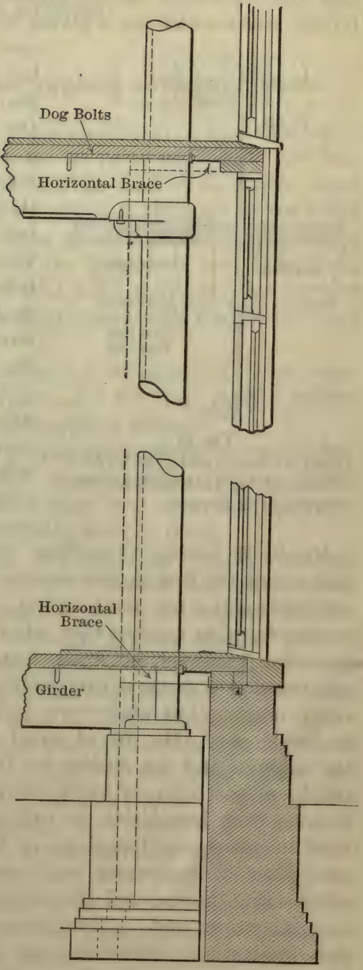

Fig. 14
Vertical Section through One Bay.

and are dog-bolted to the girders. The advantages claimed for this construction are a saving of about 33 per cent. in brick-work, with an increase in lighting of about 33 per cent., and also a saving of 10 per cent. in the height of the building, and a consequent saving in heating. The hollow piers between the bays can be used as air ducts for heating and ventilating. Five large cotton mills have been erected in the Southern States on this system of construction, which appear to verify the claims of the inventor.

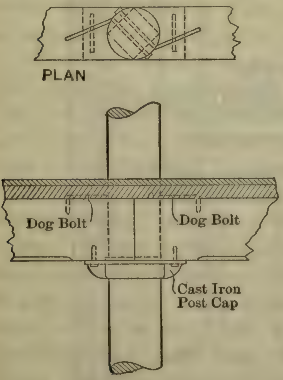

Fig. 15
Detail of Beam and Column Connections for Intermediate Supports in all Stories.

Architects or owners can make use of this system by paying a royalty to Mr. Praray, which will also include many practical suggestions as to the carrying out of the system.

Mr. S. E. Loring, Consulting Architect of Syracuse, N. Y., and one of the first in this country to advocate the use of porous terra-cotta for fire-proofing, has also patented a system of slow-burning construction, which is a form of skeleton construction executed in wood instead of steel. The interior construction is made entirely self-supporting, so that the walls carry nothing but their own weight, and may consequently be made very thin, or of wood veneered with brick. Both the columns and the girders are built up of 2-inch plank; the planks of the columns break joint so that the column is continuous from foundation to roof, and the horizontal and vertical members are joined so as to make the entire skeleton one piece of framework, and consequently very rigid. The construction is rendered slow-burning by the size of the structural members, and by interlinings of felt, asbestos, or equivalent fire-resisting materials, and also by the use of fire-proof paints, tiles, metal sheathings, etc.

A large four-story factory, at Nashville, Tenn., for the National Casket Co., was built in the fall of 1902, on this principle of construction, and there are several examples of it in New York State. Mr. Loring claims that the New England Fire

Insurance Companies have granted steel structure ratings to all buildings erected on this principle.

Mill Construction as applied to Warehouses.

The features of bad construction mentioned on page 688, are as objectionable in warehouses as in factories, while the construction advocated in mills may be used with almost equal advantage in the erection of warehouses, although as the latter are usually erected in the more thickly settled portions of a city, they are more subject to the dangers of a conflagration, and it should be understood that even the best slow-burning construction will stand but a short time after a fire has obtained a good headway.

The main object of mill, or as it is often called, "slow-burning" construction, being to prevent a fire from readily getting started, or from spreading in concealed spaces.

In applying the principles of mill construction to warehouses, therefore the general principle of using large timbers placed as far apart as the loads will permit, and of avoiding all concealed spaces, should be constantly kept in mind.

Warehouse floors, however, are generally required to sustain heavier loads than are found in woollen and cotton mills and hence require heavier construction. While warehouse floors are quite often built with transverse girders, eight or ten feet apart, with the space between spanned by flooring from four to six inches thick, the more common method of construction is to use one or more lines of longitudinal girders, supporting floor beams spaced from two to four feet apart. Where very heavy loads are to be supported this is generally the more economical construction, as it requires only a 2-inch underfloor, while it is just about as slow-burning.

Steel and Iron not as Desirable as Wood.—Wherever wooden joists and flooring are to be used, it is more desirable from the point of safety from fire to use wood for the posts and girders also, than to use iron or steel for these portions of the building, for the reason that steel beams will warp and twist and pull down the building several minutes before the wooden beams will be burnt to the breaking point, *.i.e,* provided the wooden beams have a sectional area of at least 72 square inches and are spaced 4 feet or more from centres.

Cast-iron columns will also generally fail in a fire sooner than wooden posts.

By using Oregon fir, or long-leaf Southern pine for the posts and girders and placing the posts close enough together, it is practicable to obtain sufficient strength for a five-story building with a live load of 300 pounds per square foot.

If it is thought necessary to place the posts so that the span of the girders will be more than 12 feet, then it will be necessary to use steel beams for the girders, but to obtain slow-burning construction, all steel and also all cast-iron columns should be protected to some degree from the heat.

Fire-proofing of Steel and Iron for Slow-burning Construction.—Absolute fire protection of the steel and iron is hardly warranted in a building in which the larger portion of the construction is of wood, but sufficient protection should be given that the girders and columns will stand at least until the floor timbers have fallen. Such protection for steel beam girders can be most economically obtained by first enclosing the girder with wood, then furring with $\frac{3}{4}'' \times \frac{1}{4}''$ corrugated band iron, and then covering with metal lath and plaster. The furring can be secured to the wood by a few staples, and the metal lath by nails or staples according to the kind of lathing that is used. Sheathing lath may also be used in place of the metal furring.

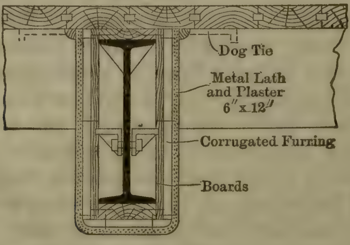

Fig. 16

Fig. 16 shows a 20″ steel beam girder protected in this way, the floor beams being $6'' \times 12''$ supported on malleable iron brackets bolted to the I-beam. Such a covering would undoubtedly protect the steel until the floor beams had dropped, and it is hardly to be expected that the columns and girders would stand after all of the floors had fallen.

The columns can be protected by metal furring and metal lath and plaster as shown in Fig. 17. For round cast-iron columns,

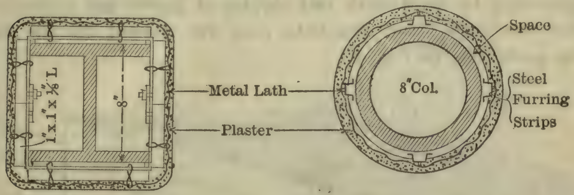

Fig. 17

Berger's economy stud, see Chapter XXIII, is probably the cheapest form of furring that can be employed, as it is easily applied and the lathing does not have to be wired to the furring.

Connection of Floor Beams and Girders.—To render the construction slow-burning, and particularly the girders, it is important that there be *no hollow space* between the top of the girders, and the flooring, or that the tops of the floor beams shall be flush with the top of the girder. This, of course, necessitates framing of the floor beams to the girder.

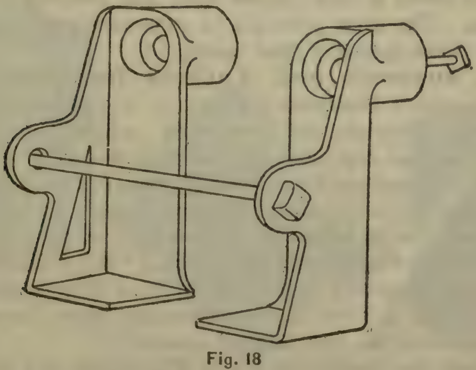

Fig. 18

For heavy construction the only kind of framing that is permissible, is by means of some form of joist hanger. The various forms of joist hangers now in the market have been illustrated and commented on in Chapter XXI. When the floor beams are $6'' \times 12''$ or larger, and the girders are of wood, the author

would give the preference to the Duplex hanger shown in Fig. 18. If the girder is of steel, the Van Dorn or National double hanger will probably be more satisfactory, as these hangers can be used with any depth of beam and girder, or special malleable iron brackets may be riveted to the web of the girder, as in Fig. 16.

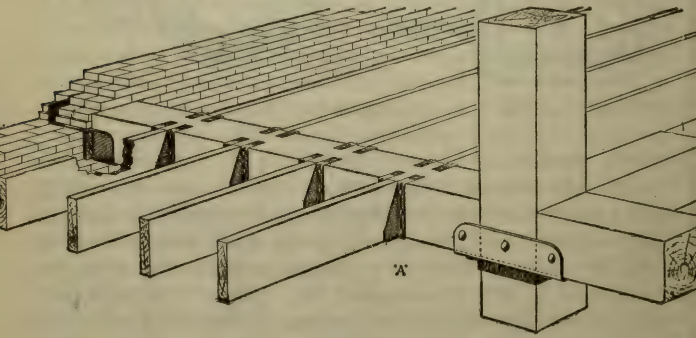

Fig. 19

Fig. 19 shows a floor framed with Van Dorn hangers and post caps. The same principle of construction is applicable to larger joists spaced further apart.

Wall Supports and Anchors for Joists and Girders.—In a warehouse intended to be constructed on the slow-burning principle, the floor beams and girders should be anchored to, and supported by the walls in such a way that in case the beams are burnt through, the ends may fall without injuring the wall, and where large timbers are used, provision should be made against the possibility of dry-rot.

The method of supporting the beams in "Mill Construction," as originally developed in the New England Mills, is shown by Fig. 20. This fulfilled the requirements above mentioned, but it weakened the wall to some extent.

The Goetz box anchors shown by Figs. 21, 22, and 23, are a decided improvement upon the anchor shown in Fig. 20 as they afford all of the advantages of the latter while they do not weaken the wall, unless the floor beams are very wide.

These anchors are made wedge-shaped so that it is impossible to pull them out of the wall, and the more weight there is upon

the beam, the greater will be the bondage that holds beam to box and box to wall.

In case of fire or accident, the joist can burn through or break, and in falling they *free* the anchorage and leave the

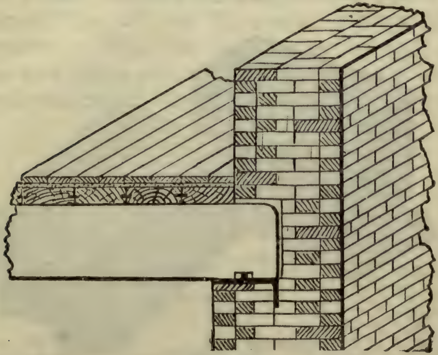

Fig. 20

wall standing, not even weakened by the space left in the wall, because the anchor remains, and the crushing strength of this

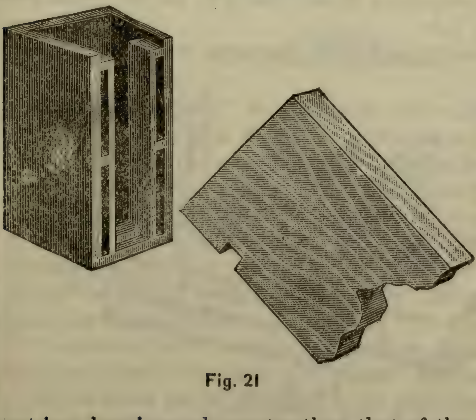

Fig. 21

Fig. 22

cast-iron box is much greater than that of the wall. No break or breach is made in the wall, and the anchor that remains, securely held, forms a space for the easy replacement of joist. The anchor provides a perfect and secure foundation for each

joist. Fire from a defective flue cannot ignite a joist end, because it is protected by a ventilated cast-iron box.

The boxes, or anchors, also have air spaces in the sides, ½ inch wide, which permit a circulation of air around the ends of the joist, effectually preventing dry-rot in the ends of the timbers.

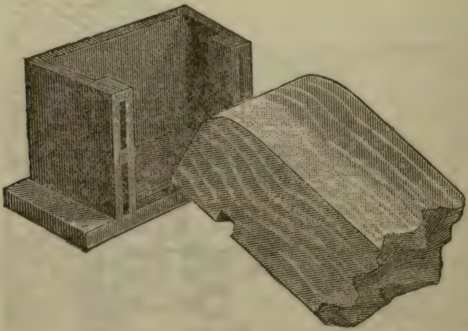

Fig. 23

If timber is wet or unseasoned it will have a chance to dry out after it is put in the building.

The average weight of a box like Fig. 22 for 2×12 joists is 10 pounds.

Another device for obtaining the same results in a different

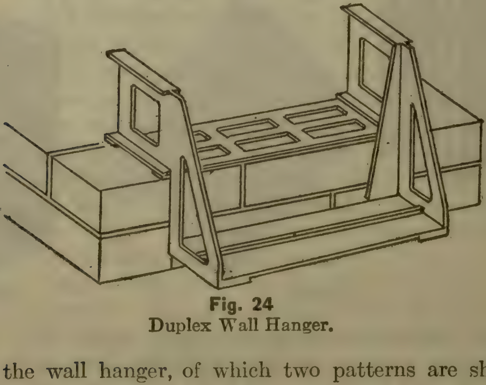

Fig. 24
Duplex Wall Hanger.

way, is the wall hanger, of which two patterns are shown in Chapter XXI. Figs. 24 and 25 show Duplex wall hangers for large timbers. The hanger shown in Fig. 25 is made extra

heavy and is provided with a plate which has eight inches bearing on the wall, and the bearing of the timbers on the hanger is also eight inches.

For beams not exceeding ten inches in breadth there is probably little choice between the box anchor, Fig. 23, and the wall hangers, Figs. 24 and 25, except perhaps in the price and appearance. When the wall hanger is used, no hole is left in the wall, and a saving of six inches in the length of the beams is effected, which in some cases would be a consideration.

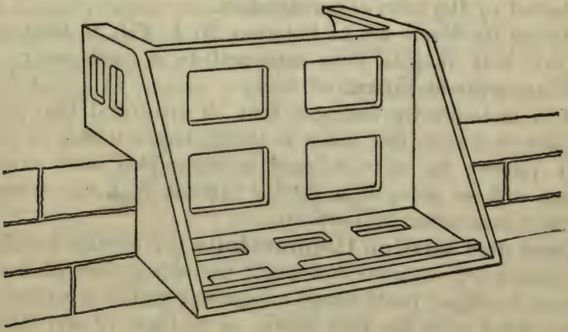

Fig. 25
Extra Heavy Duplex Hanger.

For girders 12″×14″ and upwards the author believes the hanger shown by Fig. 25 to be preferable to the box anchor. Wall hangers made from stirrups should not be used for heavy beams. Any one of these anchors or hangers is obviously greatly superior to the ordinary method of anchoring beams or girders to walls, and the use of such anchors will undoubtedly save much loss by the falling of the walls, which are almost invariably pulled down by the ordinary iron anchors when the beams fall.

A reduction in the rate of premiums for fire insurance can be obtained when these anchors or hangers are used.

Weakness of Wrought Iron Stirrups when Exposed to Fire.—Referring to this subject, Prof. J. B. Johnson, of Washington University, says:

"The recent fire tests of steel stirrups and brick walls which were made under my supervision in this city (St. Louis) show

very conclusively that unprotected stirrups are extremely dangerous. These stirrups become red hot in a few minutes and then rapidly char and burn away the ends of the beam, and also bend down so that in from twenty to thirty minutes after the fire reaches the stirrups the beam is dropped right out of the twisted steel by the straightening out of this bend or twist."

The Duplex hangers possess an advantage over steel stirrups, in that being of malleable iron they are not as quickly affected by heat, there are no twists or bends to straighten, and the bearing in the trimmer or header is to a great degree protected by the form of construction.

During the severe fire at Paterson, N. J., Feb. 9, 1902, some Duplex wall hangers were subjected to a most severe test without apparent injury.

It is undoubtedly desirable that all structural iron should be protected from fire, but it is almost impracticable to effectively protect the stirrups used in connection with wooden beams without going to a greater expense than the character of the construction will warrant.

Post and Girder Connections.—Whenever a building is constructed with wooden posts extending through several stories, the upper posts should *always* rest on top of an iron cap plate, fitted over the post below, as in Figs. 19 and 29, and never on the girder or even on a wooden bolster. A bolster would not be so objectionable but for the fact that the pressure under the post will generally be sufficient to crush the fibres of any kind of wood. Then, too, there would always be some settlement due from shrinkage. As posts are used expressly for the support of beams or girders, the iron caps must of course extend sufficiently beyond the upper post to afford ample bearing for the end of the girder. This bearing in square inches should be at least equal to one-half the load on the girder divided by the safe resistance of the wood to crushing across the grain, as given on page 414. For example, a $12'' \times 14''$ yellow pine girder is designed to support a possible load of 38,000 pounds, what bearing should it have at the ends?

Ans. The safe resistance of yellow pine to crushing across the grain is given at 500 pounds. One-half of the load on the girder is 19,000 pounds, hence the bearing area should be $19{,}000 \div 500$ or 38 square inches. As the breadth of the beam is 12 inches this would require a bearing lengthways of the girder of $3\frac{1}{6}$ inches. A bearing of 4 or 5 inches, however,

will be still better, but in no case should the bearing be less than that required by the above rule.

Forms of Post Caps.—A very common form of post cap is shown by Fig. 26, the dimensions given being for a 10×10 post. Fig. 27 shows a similar cap for a round post. These caps fulfill all requirements for strength and permit of the use of a girder wider than the post. When the girders and joists are in place, and especially when the building is occupied, there is no danger of the girders or posts slipping on the plate—in fact it would require a great force to move them. The

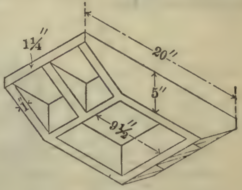

Fig. 26
Common Cast-iron Post Cap.

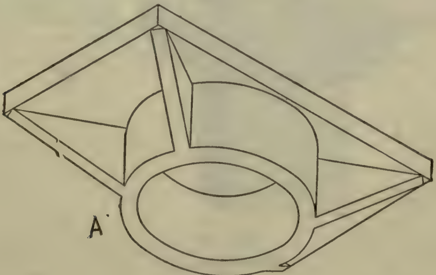

Fig. 27

girders should be tied together longitudinally by iron straps spiked to their sides.

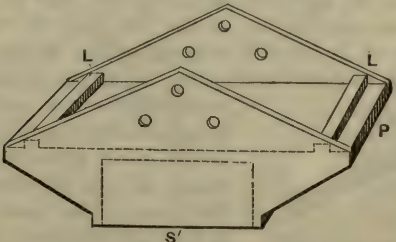

Fig. 28
Goetz Post Cap.

Many persons, however, consider it important that in a building of slow-burning construction the posts shall be tied

together in vertical lines, and the girders secured in such a way that they will be self-releasing without pulling down the posts. Figs. 28 and 29 show two post caps which fulfill these requirements.

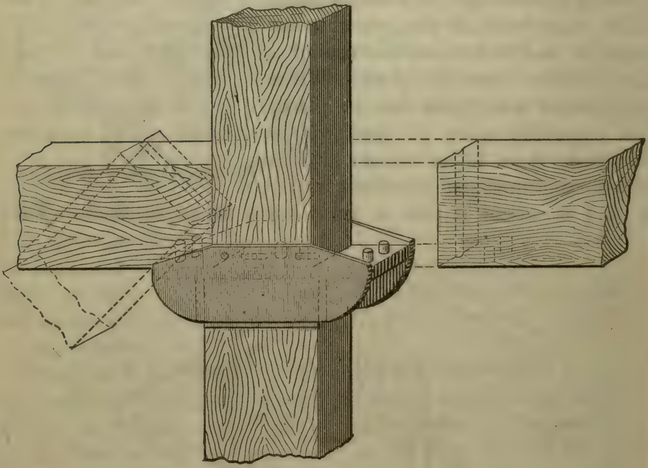

Fig. 29
Duvinage Cap.

With these caps, the ends of the girders are not fastened by bolts or spikes, but are held in place and tied longitudinally by means of the lug *L* on the Goetz cap, and by the pins on the Duvinage cap, so that in case the girder is burned to the breaking point, it can fall without pulling on the post. Provision is also made for bolting the cap to the upper post. The author doubts very much, however, if posts bolted together in this way would stand after the girders had fallen, as the planking would be likely to pull the posts over, even if they did not burn as quickly as the beams. Both of the caps shown by Figs. 28 and 29 are patented and can not be used without paying a small royalty to the patentees. A cap like that shown

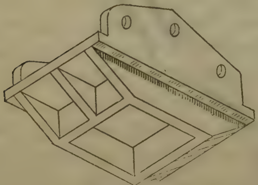

Fig. 30

by Fig. 30 without the lugs or pins can be made at any foundry without infringing on the patents.

Figs. 31 and 32 show different styles of steel caps that are largely taking the place of the cast-iron cap. The Duplex

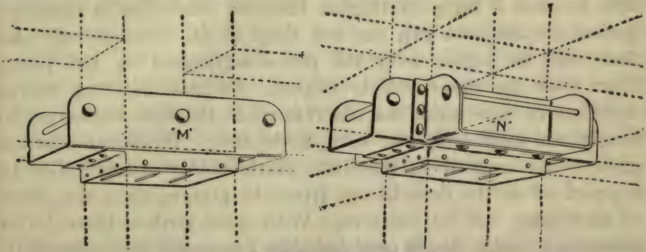

Fig. 31
Van Dorn Post Caps.

post caps are also made so as to permit of the extension of the post for two stories, thereby giving an extremely strong and

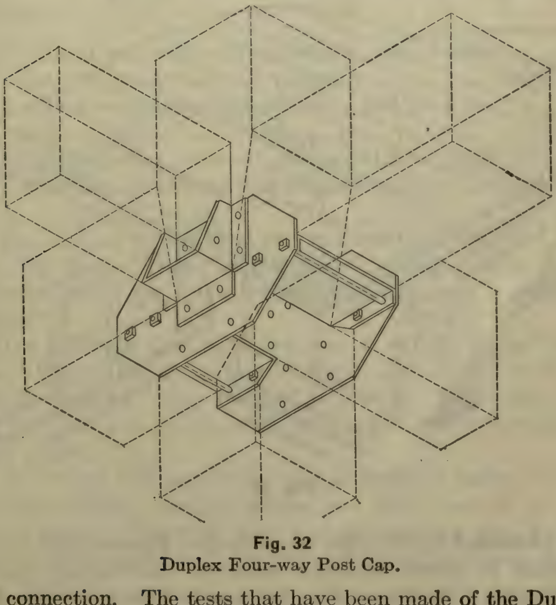

Fig. 32
Duplex Four-way Post Cap.

stiff connection. The tests that have been made of the Duplex post caps show that they possess great strength.

There is an objection to the use of four-way post caps where the girders are of wood, in that the floor beams that are hung from the girders will drop by an amount equal to the shrinkage in the girder, if the beams are hung in stirrups, or by one-half this amount if hung in Duplex hangers, while the beams supported on the post cap can not drop at all, consequently the floor will be higher over the beam supported by the posts, than over the intermediate beams. In one building where deep beams were used, this unevenness in the floor amounted to nearly an inch and was very noticeable. Wherever wooden girders are used it is therefore much better construction to support *all* of the floor beams from the girders, then the effect of shrinkage will be uniform. With steel girders there is no shrinkage, and a beam may be placed opposite the posts with advantage.

Mill Construction with Concrete Flooring.— Fig. 33 shows a modification of mill or slow-burning construc-

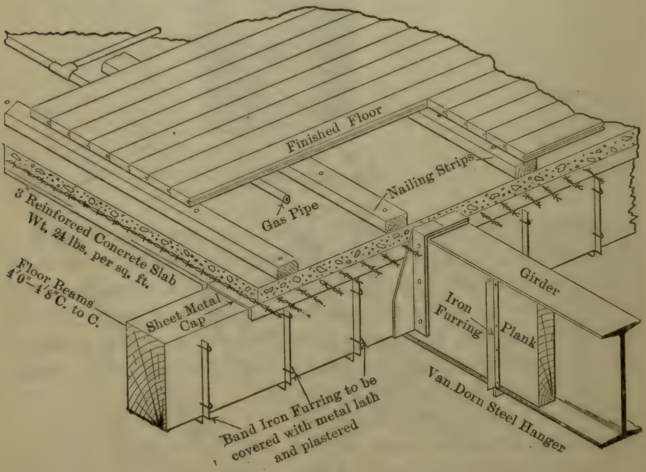

Fig. 33

tion advocated by the Hinchman-Renton Fire-proofing Co for buildings in which first-class fire-proof construction cannot be afforded. It differs from the standard slow-burning construction only in the substitution of reinforced cinder concrete

for the usual 2 or 3-inch plank between the beams. While this construction has never been tested by a fire, it appeals to the author as an improvement upon the standard wooden construction, in that there is less wood to burn, while none whatever (except the finished flooring) is exposed, and it is more sound proof and more decorative than a floor ceiling without the plaster. Were it not for the element of shrinkage which can never be entirely overcome where wood is used for floor beams, the author believes that this floor would stand fire fully as long as many so called fire-proof floors with steel joists. This floor can also be finished on top with cement.

Cost of Mills and Factories Built on the Slow-Burning Principle.

The cost per square foot of total floor area of mills and factories in the year 1884 was, according to Mr. Edward Atkinson, as follows:

Mill with three stories for machinery, and a basement for miscellaneous purposes............75 to 80 cts.
Mill with two stories for machinery, and no basement.. 65 cts.
Mill with one story, of about one acre of floor, with basement for heating and drainage only......about 85 cts.

The above is for the total area of floors in the building, above the basement. These figures, while perhaps a little low for present prices, are a fair average taken through a number of years.

The following data as to the cost of mill buildings built on the "Praray Improved Construction" and also of mills built on the modern "Slow-Burning Method of Construction" both in the East and South, were kindly furnished the author by Mr. Praray.

Mr. Praray also says: "The cost of building varies according to the locality. Lumber, bricks, and labor cost 50 per cent. more in New England than in the Carolinas, and therefore the same building will cost much less in the South than in the East."

Examples.—The Dixie Cotton Mills at La Grange, Ga.,

built in 1895 and 1896 on what is called the "Praray Improved Construction."

Total floor area of all buildings and steam plant, 87,100 square feet. Cubical contents of all buildings, including the Smoke Stack, which is 125 feet high, 52″ core, 2,419,000 cubic feet.

Cost, per square foot floor space, $43\frac{63}{100}$ cents.

The Whitman Cotton Mills, New Bedford, Mass., built in 1896 and 1897 on Modern Mill Construction, and approved by the Mfrs. Mutual Ins. Companies.

Total floor area of all buildings and steam plant, 217,000 feet. Cubical contents of all buildings, including smoke stack, which is 200 feet high, 10 feet core, is 6,494,000 cubic feet.

Cost per square foot floor space, $81\frac{19}{100}$ cents.

The Moorhead Cotton Mills, Moorhead, Miss., built in 1900 on what is called the Modern or Slow-Burning Construction, and higher above grade than is usual on account of high-water mark.

Total floor area of all buildings and steam plant, 31,000 feet. Cubical contents of all buildings, including the smoke stack, which is 125 feet high, 48″ core, 532,640 cubic feet.

Cost per square foot floor space, 82 cents.

The above three mills were built for cotton mills, and are all equipped with steam-power plants.

All are two-story buildings. It will be noticed that the Mississippi mill cost more per foot floor space than the New England mill.

The Georgia or Dixie mill cost about one-half per foot floor space of the two other mills.

Five other mills on the "Praray Method" have been built in the South, and the cost has been less than 50 cents per foot floor space.

Mills built throughout the New England States cost from 75 cents to $1.10 per square foot floor surface

Mills built in the Carolinas, Georgia, and Alabama average about 60 cents per square foot floor surface.

Mills of less than 100,000 square feet floor surface will cost from 80 cents to $1.25 per foot.

The greater amount of floor space will enable mills to be built on the modern methods at the prices named above.

Labor in the South is much cheaper and surely 33 per cent. slower. Lumber and materials in the South cost about 50 per cent. of the price paid in New England.

A mill built at Taunton, Mass., four stories high with a floor area the same as the Whitman mills, cost 96 cents per square foot floor space. This was also equipped with steam power.

The cost per square foot of floor space will generally be greater for a small mill than for a large one for the reason that a small mill that is to be run by steam power has to build boiler and engine house, pump house, towers for tank purposes, etc., large enough for doubling the size of the mill at some future time.

The mills in the South are built for from 5,000 to 10,000 spindles to start with, and are planned to add that amount of spindles at some future time. They must build boiler and engine house, belt-ways, towers, etc., to accommodate future extensions, which makes the first cost of the mill come very high, and very often places the mill in an embarrassing position.

For example, the Moorhead Cotton Mills, Moorhead, Miss., was built with the idea of adding double the amount of spindles that they have installed, that is to say, the power house and chimney were all large enough for the future extension, making spare room in the boiler house, and spare room in the engine house.

CHAPTER XXIII.

FIRE-PROOFING OF BUILDINGS.

By Rudolph P. Miller, C. E.

Definitions.—The term "fire-proof," while now quite well understood by architects, is still used in a very broad sense by the public. To be strictly fire-proof, a building must be constructed and finished entirely with incombustible materials, and any of these materials, such as steel or iron, which are injuriously affected by heat or streams of water must be efficiently protected by other materials which are not so affected.

This precludes the use of wood, whether exposed or not exposed, also all exposed steel or iron, common glass, and most building stones.

It is safe to say that there are very few buildings in this country that are absolutely fire-proof—there is quite a large number, however, that could not be destroyed by fire, and in which the salvage would probably amount to from 60 to 80 per cent., and it is the latter class which is generally meant when the term fire-proof is used.

Incombustible buildings, and buildings having wood construction protected to a greater or less degree from the flames, are sometimes advertised as fire-proof, but such buildings should be considered merely as slow-burning.

To build absolutely fire-proof is expensive—to build so that the building can be merely gutted by fire, is less expensive, and for many classes of buildings all that is advisable.

It is undoubtedly the duty of every architect to be well informed concerning the fire-proof qualities of all materials that enter into the construction and finishing of buildings, and as to the best use of these materials—he can then choose his materials and use them to such an extent as the character of the building and the interests of his client demand. It is this

information that this chapter is intended to furnish in a concise manner.

Municipal Definitions. — Municipal definitions as to what constitutes "fire-proof construction" have a great bearing on the construction of buildings within their jurisdiction, and those of the two largest cities are therefore quoted.

Chicago Definition.—"The term fire-proof construction shall apply to all buildings in which all parts that carry weights or resist strains, and also all exterior walls and all interior walls and all interior partitions and all stairways and all elevator enclosures are made entirely of incombustible material, and in which all metallic structural members are protected against the effects of fire by coverings of a material which shall be entirely incombustible, and a slow heat conductor, and hereinafter termed 'fire-proof material.' Reinforced concrete as defined in this ordinance shall be considered fire-proof construction.

"The materials which shall be considered as filling the conditions of fire-proof covering are: First, burned brick; second, tiles of burned clay; third, approved cement concrete; fourth, terra-cotta; fifth, approved cinder concrete."

New York Definition.—Buildings required to be fire-proof shall be "constructed with walls of brick, stone, Portland cement concrete, iron or steel, in which wood beams or lintels shall not be placed, and in which the floors and roofs shall be of materials provided for in Section 106 of this Code. The stairs and staircase landings shall be built entirely of brick, stone, Portland cement concrete, iron or steel. No woodwork or other inflammable material shall be used in any of the partitions, furrings, or ceilings in any such fire-proof buildings, excepting, however, that when the height of the building does not exceed twelve stories nor more than 150 feet, the doors and windows and their frames, the trims, the casings, the interior finish when filled solid at the back with fire-proof material, and the floor-boards and sleepers directly thereunder, may be of wood, but the space between the sleepers shall be solidly filled with fire-proof materials and extend up to the underside of the floor-boards.

"When the height of a fire-proof building exceeds twelve stories, or more than 150 feet, the floor surfaces shall be of stone cement, rock asphalt, tiling, or similar incombustible material, or the sleepers and floors may be of wood treated by

some process approved by the Board of Buildings, to render the same fireproof. All outside window frames and sash shall be of metal, or of wood covered with metal. The inside window-frames and sash, doors, trim, and other interior finish may be of wood covered with metal, or of wood treated by some process approved by the Board of Buildings to render the same fire-proof."

Section 106 refers to fire-proof floors. These may be constructed of brick, tile, cement concrete, and, in fact, of any material that will successfully pass the tests prescribed by the Code. Before any floor construction other than brick or tile will be passed by the department, however, it must be tested for strength and fire resistance under very rigid conditions, and the construction must stand the test successfully.*

When Fire-proof Construction Should be Employed.—A building should be designed, built, and finished to conform to the purpose for which it is to be used—a building that will contain but little inflammable material, and that not of great value—need not be as thoroughly fire-proof as a building designed for the storage of valuable goods, or where the safety of human life is at stake.

The height of a building is an important factor in determining whether it should be fire-proof or not. The ability to cope with fire in a building increases in more than direct proportion to its height. The area covered by a building also is important, although in most instances interior division walls may be provided which practically cut up a building into a series of smaller buildings.

Some of the limitations placed on non-fire-proof buildings by various municipal laws will be found in the table on page 729.

Limiting Areas for Non-fire-proof Buildings.—

New York,	8,000	sq. ft. on an interior lot.
	12,500	" " " a corner.
	22,000	" " when facing three streets.
Chicago,†	9,000	" " if of ordinary joisted construction.
	12,000	" " if of slow-burning construction.
St. Louis,	7,500	" "
Boston,	5,000	" "

* See p. 745.

† Milwaukee has same law.

Cleveland, Mill Construction.*

20,000 sq. ft. when facing streets on four sides.
15,000 " " " " " " three "
12,000 " " " " " " two "
9,000 " " " " " " one side.
5,000 " " on any lot when of hazardous occupancy.

Ordinary Construction:

12,500 sq. ft. when facing streets on four sides.
10,000 " " " " " " three "
7,500 " " " " " " two "
5,000 " " " " " " one side
2,000 " " on any lot when of hazardous occupancy.

LIMITING HEIGHTS FOR NON-FIRE-PROOF BUILDINGS

City.	All Buildings.	Hotels.	Schools.	Hospitals and Asylums.	Residence Buildings.
Buffalo, N. Y.	72 ft.				
New York, N. Y.	75 ft.	35 ft.	35 ft.	35 ft.	
Kansas City, Mo.	75 ft.				
St. Louis, Mo.	75 ft	.....	2 stories above basement	2 stories above basement	4 stories and basement
Boston, Mass.	75 ft.	....	....	....	65 ft.
Washington, D. C.	75 ft.	60 ft.	....	....	60 ft.
New Haven, Ct.	75 ft.	60 ft.	60 ft.	45 ft.	
Denver, Col.	80 ft.	....	3 stories	3 stories	
San Francisco, Cal.	84 ft.				
Seattle, Wash.	85 ft.	3 stories	....	....	3 stories
Omaha, Neb	90 ft.				
Chicago, Ill.	100 ft.	....	3 stories	2 stories	5 stories and basement
Baltimore, Md.	100 ft.				
Milwaukee, Wis.	100 ft.				
Cleveland, O	100 ft.				
New Orleans, La.	......	52 ft.	....	52 ft.	
Newark, N. J.	......	....	....	....	3 stories
Philadelphia, Pa.	......	....	....	....	4 stories

Cost of Fire-proof Construction.—Mr. F. W. Fitzpatrick, consulting architect, contributed to the March, June, and July, 1903, numbers of *Fireproof* some interesting papers in which he gives the comparative cost per cubic foot of a large number of buildings figured for both fire-proof and ordinary construction. These figures seem to indicate that fire-proof construction for office-buildings, hotels, etc., adds from 9 to

* See Chap. XXII.

13 per cent over the cost of ordinary construction with wooden joists. For stores and warehouses the difference will often be less than 5 per cent.

See also, "Cost of Buildings per cubic foot," Part III.

Divisions of the Subject.—In constructing fire-proof buildings it is necessary to consider:

1. *Materials* to be used.
2. *Form of construction.*
3. *Protecting devices.*
4. *Extinguishing appliances.*

This arrangement will be followed in the discussion of the subject in this chapter.

Materials of Construction.

All materials of construction are more or less injuriously affected by high temperatures. Furthermore, an incombustible material is not necessarily fire-resisting, as, for instance, steel. The value of various materials in fire-proof construction is indicated in the following paragraphs.

Brickwork.—Common brickwork, when of a good quality, will stand exposure to fire for a considerable length of time, but in a severe conflagration the heated side of the wall expands, often to the point of throwing the wall, the bricks crack, shell, and are sometimes melted. Experience has shown that thick walls are less affected by heat than thin walls, and that hard-burned bricks will stand better than soft or under-burned bricks. In buildings which are to contain large quantities of inflammable material it is undoubtedly better to line the walls with porous furring tile or hollow brick.

In the Baltimore and San Francisco fires, it was demonstrated that for outside walls brick is superior as a fire-proof material to any other material used in wall construction.

Stones.—Very few stones will successfully stand the action of severe heat, and consequently stone should be used very sparingly in fire-proof buildings, and certain stones not at all.

Granite will explode and fly to pieces or disintegrate into sand when exposed to flames.

Limestones and Marbles are usually ruined if not totally destroyed by an ordinary fire. They are the least desirable of all stones to use in a fire-proof building, and the granites come next.

Sandstones when fine-grained and compact will sometimes stand fire without serious injury, but in the case of a severe conflagration are generally so badly affected that they have to be replaced.

Terra-cotta is made from clay by mixing with water into a plastic mass, shaping the same into the form and shape desired and baking at high temperature in kilns. For the usual structural form the shaping is generally done by forcing the plastic mass through a special die by means of machinery. Ornamental terra-cotta must generally be shaped by hand.

Ornamental Terra-cotta. — This is undoubtedly the best material to use for the trimmings of a building that is intended to be absolutely fire-proof, and especially that which has a glazed surface. "Terra-cotta will certainly require less reconstruction after severe fire-and-water tests than any building material, saving, possibly, the best qualities of fire-brick." *

Structural Terra-cotta. — Terra-cotta, as used for floor arches, column and girder protection, and for building light hollow walls, is made of three different compositions, the material being known as "Dense," "Porous," and "Semi-porous," according to the method of manufacture.

Dense-Tiling is made from a variety of clays. Some manufacturers use more or less fire-clay, and combine it with potter's clay, plastic clays, or tough brick clay. It is very dense, and possesses high crushing strength. In outer walls exposed to the weather and required to be light, it is very desirable. Some manufacturers furnish it with a semi-glazed surface for outer walls of buildings. For such use it has great durability, and effectually stops moisture. In using dense tiling for fire-proof filling, care should be taken that the tiles are free from cracks, and sound and hard burnt.

Porous and Semi-porous Terra-cotta is made by mixing sawdust with the clay, the sawdust being destroyed by the action of the heat, leaving the material light and porous. A small proportion of fire-clay mixed with the plastic clay is desirable but not essential. The proportion of sawdust should be from 25 to 35 per cent, according to the toughness of the clay used. Care is required in manufacture that the work of mixing, drying, and burning be thoroughly done. The burning

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* Freitag.

should be done in down-draught kilns by a quick process. The product should be compact, tough, and hard, ringing when struck with metal. Poorly mixed, pressed, or burned tiles, or tile from short or sandy clays, present a ragged, soft, and crumbly appearance, and are not desirable. When properly made, porous terra-cotta will not crack or break from unequal heating, or from being suddenly cooled with water when in a heated condition. It can be cut with a saw or edge tools, and nails or screws may be easily driven into it for securing interior finish, slates, tiles, etc. For the successful resistance of heat, and as a non-conductor for the protection of other materials, it must be ranked among the very best.

Semi-porous Tiling. — This material was introduced by those factories which use pure fire-clay in the manufacture of their tile, to enable them to compete with the standard porous material.

During the process of grinding the clay about twenty per cent of ground coal is mixed with it. This coal aids in the burning of the material and also makes it lighter and more or less porous. Tiling made by this process is admitted to be a much better fire-resistant than the solid or dense material.

Mr. E. V. Johnson says: "Personally, I believe that good semi-porous fire-clay tile is fully as efficient as a fire-resisting material as the standard makes of porous terra-cotta."

Strength of Terra-cotta.—In a series of tests recently made at Columbia University for the building authorities of New York on terra-cotta blocks taken from material delivered in the open market, the following crushing strength was developed:

Dense tile 5820 lbs. per square inch (an average of 10 tests).

Semi-porous terra-cotta 3292 lbs. per square inch (an average of 10 tests).

The inequality in strength of the two materials can be overcome by using thicker webs and shells for the semi-porous or porous material.

In the matter of weight, porous and semi-porous terra-cotta have the advantage over dense tile. It has already been pointed out that porous terra-cotta is more readily cut and will hold nails or other fastenings which can be driven into it without destroying the material.

Dense tiling, when heated and cooled by water, is liable to crack from the sudden contraction; "blocks with two or more

air-spaces are very liable to have the outer webs destroyed under this action. Even if not cooled with water, other fires have shown that hard-burned terra-cotta will crack and fall to pieces under severe heat alone." *

The experience of the recent conflagrations in Baltimore and San Francisco fully bears out this statement. The collapse of the floors of one of the buildings in Baltimore was largely due to the weakening of the terra-cotta arches by reason of the breaking off of the outer shells.

Porous terra-cotta is non-heat-conducting in itself, and the best qualities will usually resist fire and water successfully, but if the product "is not burned at a sufficiently high temperature to consume all of the sawdust, the throwing of cold water upon the heated surfaces will cause an expansion or disintegration due to the absorption of the water and its conversion into steam."

Porous terra-cotta absorbs water freely, and if allowed to freeze when wet is more or less injured. If the process is permitted to continue, the blocks become so weakened as to make them unsafe for use.

Concrete.—Stone concrete, under the action of heat, is affected much the same way as brickwork. The heated surface expands and, as the concrete is a very poor conductor, the other surface remains cool and either cracks or causes warping. The heat also affects the strength and texture of the concrete, causing a disintegration of the concrete to a depth of about one inch. Often the surface spalls off with more or less of a report. If water is applied after the heat, the surface is washed away to the depth of the affected part. These effects vary somewhat with the stone used in the aggregate. Gravel and granite, on account of difference in coefficient of expansion between them and concrete, are likely to spall. Limestone calcines under the action of heat and is especially liable to destruction for some depth by the water. Trap-rock is the most satisfactory material to use, from the fire-resisting standpoint as well as that of strength.

If there is no application of water after the fire and the surface is allowed to cool off gradually, the concrete may set again and become hard. It is not well, however, to rely on this.

Slag Concrete.—Blast-furnace slag has been used as the

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* Freitag.

aggregate in concrete, with very satisfactory results both as to fire-resistance and strength.

Cinder Concrete.—Cinder concrete, because of its porous character and the nature of its aggregate, makes a most excellent fire-proofing material. Tests and the experience of recent conflagrations would indicate that it is the best. Care must, however, be taken in the selection of the cinders. They must be clean furnace cinders, free from particles of unburnt coal, and should be ground by machinery before mixing for concrete.

When properly selected and proportioned cinder will produce good concrete, but generally a very ununiform material is obtained, so that its strength is variable and doubtful. For this reason in using cinder concrete in floor construction the working loads should be determined from load tests and a high factor of safety should be used. The practice in New York istotake one-tenth of the breaking load as the working load.

Corrosive Action of Cinders.—When cinder concrete is used to encase steel, either as a protective covering or as a part of a concrete construction, the corrosive effect of cinders must be guarded against.

A discussion of this subject will be found in the following chapter, p. 882*d*.

Mortars, Plasters, and Plaster of Paris.—Mortar and plaster must necessarily enter into the composition of all masonry buildings, whether built of brick, stone, or terra-cotta. That ordinary lime mortar, when well made, will endure for unlimited periods of time, in dry situations, has been proven by actual use.

Hydraulic cement mortars are equally durable in wet or damp places.

For laying brick or tile work in first-class buildings, cement and sand mortar is preferable to any other, and cement mixed with lime mortar gives greater strength than lime and sand alone.

Regarding the fire-proof qualities of mortars and plaster compositions there has been much controversy; the truth of the matter seems to be that all such compositions will withstand the action of heat up to a certain degree, when they are affected in one way or another depending not only upon the composition but in a large degree upon their body, and upon the way in which they are used.

Lime mortar for walls was formerly considered as the most satisfactory, so far as fire-resistance is concerned, but since the improvements in cement manufacture, cement mortar is generally preferred.

Lime plaster, applied on wire lath, will withstand a high degree of heat without injury, but is liable to be washed away in places by streams of water.

Hard wall plasters, or patent plasters, when applied to brick work or metal lath, are in all cases equal in heat-resistance to common lime, and many of the patent plasters will stand the combined effects of fire and water longer than the common mortars.

Plaster of Paris.—Compositions of plaster of Paris and broken bricks, wood chips, or sawdust are non-conductors of heat and possess fire-resisting properties of considerable magnitude, and, on account of their lightness and cheapness, are used to quite an extent in fire-proof or semi-fire-proof buildings.

In France such compositions have been used for generations for forming ceilings between beams, and its durability and fire-proofing qualities are there unquestioned.

Plaster-of-Paris compositions when subjected to a severe heat are softened on the surface, and when water is thrown upon it the plaster washes away to some extent.

Asbestic Plaster. — A plaster made by mixing asbestic with freshly slacked lime-putty has been used to some extent in New York City. Asbestic is made from a serpentine rock, mined near Montreal, which contains a large proportion of asbestos.

"Claims of great fire-resisting properties are made for this material, as well as resistance to the effects of water during fire; cracking and discoloration due to the percolation of water or acids are also claimed to be avoided. The plaster is tough and elastic, and it will receive nails without chipping or cracking. The weight is said to be about half that of ordinary cement mortar."

Asbestic was subjected to a severe fire-and-water test in the presence of the officials of the Supervising Architect's office at Washington, "and the plaster did not crack or drop, but remained intact."

"All of the walls, ceilings, and columns of the appraiser's warehouse in New York City were covered with a coat of asbestic

applied from ½ to ¾ of an inch thick, on the concrete or terra-cotta surfaces.

"The great objection to the use of this material lies in its slow drying, the time required for a thorough drying out being usually very long." *

Steel and Wrought Iron.—Wrought iron and steel will expand, bend and twist under a moderate degree of heat. Inasmuch as a temperature of 1700° F. is not unusual in fires, these materails should not be used in fire-proof construction without proper protection.

Fire tests at the Continental Iron Works in 1896 showed that unprotected steel columns under load began to fail when the temperature had reached about 1100° F.† In the Baltimore and San Francisco fires there were many instances of failure in steel columns due to lack of or to insufficient protection.

Cast Iron.—"As the result of tests and actual experience in conflagrations it may be stated that unprotected cast iron can stand practically unharmed up to temperatures of 1300 or 1500° F. while carrying very heavy loads even with frequent applications of cold water while the metal is at a red heat." * In the tests at the Continental Iron Works, referred to in the previous paragraph, a temperature of nearly 1300° F. was reached before the cast iron columns began to fail.

The contents of most mercantile buildings, when burning freely, would probably generate a heat exceeding at times 2000 degrees. Consequently cast-iron columns, when unprotected, are almost sure to fail in such a fire either by bending or breaking.

No building can be considered fire-proof in which unprotected iron or steel columns are used, but in many classes of buildings unprotected cast-iron columns might safely withstand any heat to which they would probably be exposed. From a fire-resisting point of view, cast-iron columns are unquestionably preferable to steel columns when unprotected.

Fire-proof Wood.—To meet a demand for an incombustible flooring in the warships of the U. S. Government, and the requirements of certain provisions of the New York Building Code, an attempt has been made to produce fire-proof wood.

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* Freitag.

† See *Engineering News*, Aug. 6. 1896.

The processes for rendering wood fire-proof, in general, consist in impregnating its fibers with certain chemicals. After the fire-proofing process, the lumber should be thoroughly kiln-dried before it is allowed to enter into construction. The kiln-drying, to be done properly, requires several weeks, the actual time varying in accordance with the thickness of the stock, but to secure a first-class job in fire-proofed wood it is essential that the stock be bone-dry.

The soft woods are more easily, thoroughly treated than the hard woods, the resinous woods being particularly difficult to handle.

"The treatment of the wood to render it fire-proof slightly raises the igniting-point of the wood. The treated wood is harder to light than the untreated wood, taking two to three times as long to ignite. The amount of wood destroyed when exposed to the action of a flame is from 5 to 12 per cent greater in the case of an untreated wood than in the case of a treated wood. The untreated wood furnished more flame than the treated wood. The untreated wood will sustain flame longer than the treated wood after the source of heat has been removed. From this it can be seen that the fire-proofed wood is less likely to ignite and less likely to cause the spread of fire than the untreated wood." *

Disadvantages.—Among the disadvantages of fire-proof wood should be mentioned an increased difficulty in working the wood, a tendency to dull woodworking tools more rapidly than untreated wood. Hence an increased cost in the use of fire-proof wood. Professor Woolson estimates this increase to be from $35 to $65 per thousand feet, the hard wood costing the most.

The salts used in the process of fire-proofing being hygroscopic tend to keep the woodwork damp. Hardware or other metal work in contact with fire-proofed wood is liable to corrode. The strength of the wood is often affected, and in some cases the wood has become quite brittle. These two last-mentioned faults can be largely overcome by neutralizing the fire-proofing solution by a proper mixture of acid and alkaline salts.

Test.—The test known as the timber test applied to fire-proof wood in New York City consists in placing a stick of the treated wood, ¾ inch by 1½ inches in cross-section and 8 inches long,

* See *Insurance Engineering*, Vol. IV, p. 551; also Report No. 1 of Professor Norton to Boston Manufacturers' Mutual Fire Ins. Co.

for two minutes over a crucible gas furnace in which a constant temperature of 1700° F. is maintained, then removing the test piece, noting the time it continues to flame and glow, then scraping away the charred wood and determining the percentage of unburned wood. The conditions of acceptance are that, "the flame and glow should disappear within ten to twenty seconds after the removal of the test-piece from the furnace, and the unburned and uncharred section at the center of the specimen should be not less than 50 to 70 per cent of the original cross-section, depending on the variety of wood under test."

If the wood has been thoroughly treated, a splinter of it after having been exposed to flame and withdrawn will show no glow or flame. Other tests have been suggested and used, but need not be described here.

At the present time fire-proof treatment of wood is being done by The American Wood Fire-proofing Company, The Fire-proofine Manufacturing Company, and the Electric Fire-proofing Company, all of New York.

Wire Glass.—The introduction of this material has made it possible to secure fire protection in many cases, without the necessity of disfigurement due to fire-shutters.

Wire glass is either "ribbed," "rough," "maze," or polished plate having wire imbedded in its centre during the process of manufacture.

"The temperature at which the wire is imbedded in the glass insures adhesion between the metallic netting and the glass, and the two materials become one and inseparable, so that if the glass is broken by shock, by intense heat, or from other cause, it remains intact." It is this property of remaining intact that gives it its fire-retarding qualities, as although fire and water may cause cracks to spread throughout the glass the wire holds the pieces so firmly that flames cannot pass through. Many severe tests during actual fires have positively demonstrated the truth of the above claim. For warehouses and factories the "ribbed" or "maze" glass will generally be preferable, but for offices, or wherever clear transparent glass is desired, the "polished plate" is nearly if not quite as acceptable as the same glass without the wire, the effect being the same as looking through a window with a screen on the outside.

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* See also p 1418.

The wire netting used for this purpose is similar to the ordinary "chicken netting" with about a 1-inch mesh. The Mississippi Glass Company of St. Louis is the chief manufacturer of wire glass in this country, the material being handled by the leading glass merchants in all the large cities of the country.

Fire-proof Paint.—Numerous so-called fire-proof paints have been introduced in recent years. When applied to woodwork they provide a more or less effective protection against fire and may, for this reason, prevent the spread of fire. The Bureau of Buildings of New York makes the following regulations regarding fire-proof paint:

"First. The term 'fire-proof paint' shall be understood to mean any preparation used to cover the surfaces of wood or other materials for the purpose of protecting the same against ignition.

"Second. No fire-proof paint will be considered satisfactory unless it so protects the wood or other material to which it is applied that the same will not flame or glow after having been subjected to the flame of a gasoline torch for two minutes.

"Third. Before applying fire-proof paint to any material the surfaces must be cleaned.

"Fourth. Application of fire-proof paint must be repeated whenever it is found that the material to which it is applied is no longer protected to fulfill Specification No. 2." *

Forms of Construction.

Girder and Column Protection.—As the columns and girders of a building form the "back-bone" of the structure, it is of vital importance that they be very thoroughly protected from heat. As a rule, the manner of protecting these structural elements depends quite largely upon the floor system adopted. The concrete companies naturally prefer to use concrete wherever possible, and hence where concrete is used for the floor-construction it is generally employed for encasing the columns and girders. Where hollow tile is used in the floors, the same material is almost invariably employed for protecting the steel frame.

The methods used for protecting girders are described under the discussion on floor-construction, pp. 795 to 799.

* Annual Report Bureau of Buildings, Borough of Manhattan, N. Y., for 1904.

Necessity for Column Protection.—It is now generally recognized that iron and steel columns should be incased with some material that will thoroughly protect the metal against fire.

In 1896 a committee of the American Society of Mechanical Engineers, in conjunction with representatives from other organizations, made a series of fire tests on full-sized unprotected cast iron, and steel columns, loaded to their figured safe capacities. These tests showed that the steel columns failed at an average temperature of 1150° F., and the cast iron at an average temperature of 1300° F., the failure setting in after exposure to the fire of from 23 minutes to 1 hour and 20 minutes, or an average duration of about 50 minutes.

For the purposes of determining the value of several materials as satisfactory protective coverings, the Bureau of Buildings of New York made a series of tests on the heat conductivity of these materials. A cast-iron plate covered with the material under test was subjected to a temperature of 1700° F. for two hours over a crucible furnace, and the heat of the plate noted at regular intervals of time. The results of the tests is shown in the following table:

Material Under Test.	Temp. on Face of Protective Material. ° Fahr.	Temperature of Plate at Back of Protective Material (Degrees Fahr.)		
		Before Heating.	After Heating, 2 Hrs.	Heat Transmission.
Terra-cotta: Dense, hollow, 2″ thick	1700	75	223	148
Terra-cotta: Semi-porous, solid, 2″ thick	1700	73	244	171
Plaster of Paris and shavings, 2″ thick	1700	69	159	90
Plaster of Paris and asbestos, 2″ thick	1700	70	163	93
Plaster of Paris, wood fibres, and infusorial earth. 2″ thick	1700	72	167	95
Concrete of Ground Cinders, $1\frac{5}{16}$″ thick	1700	73	363	290
Cinder Concrete, on metal lath, 2″ thick	1700	66	248	182
Metal Lath and patent plaster, about ½″-thick over 1″ air-space	1700	76	296	218

Terra-cotta Column Protection.—Fig. 1 shows the manner in which Z-bar columns are protected in the best class of fire-proof buildings (when tile fire-proofing is used). Figs. 2, 3, and 4 show common methods of protecting round columns, and Figs. 5 and 6 square columns.

The steel guard, shown in Fig. 1, is often employed in mer-

cantile or manufacturing buildings to a height of four or five feet above the floor.

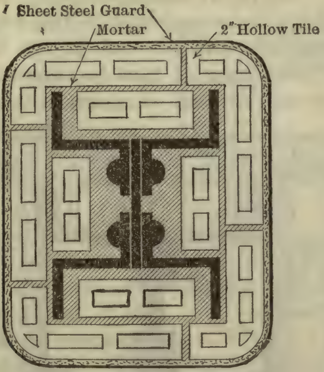

Fig. 1.

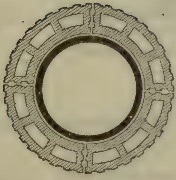

Fig. 2.

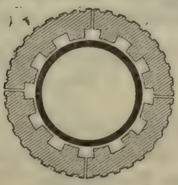

Fig. 3.

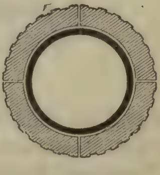

Fig. 4.

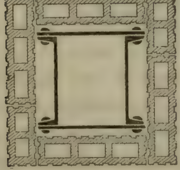

Fig. 5.

Fig. 6.

Although it is not customary to do so, the efficiency of this construction is greatly increased by wrapping the column with wire lath before plastering.

To insure the protection of the metal, under the most trying conditions, it is imperative that the protective covering shall not be detached by the streams from the firemen's hose, so as

to expose the steel. This can be positively guarded against only by using two layers of tiling or concrete and wrapping the inner layer with metal lathing.

Fig. 7 shows a column protected in this way, the construction being essentially that adopted in the Fair Building in Chicago. The inner layer of tiles is wrapped with wire lath imbedded in the mortar, and all spaces between the tiles and

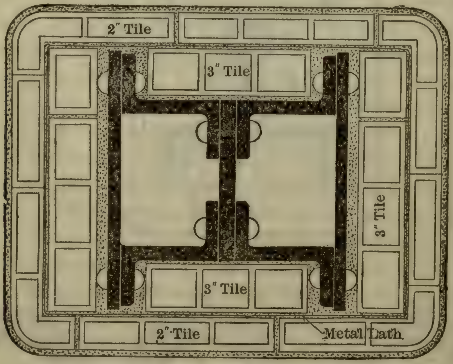

FIG. 7.

metal filled solid with cement mortar. The protection afforded by this construction should be perfect.

Concrete Column Protection.—Where concrete is to be used for column protection, the most efficient construction is undoubtedly to surround the metal with cinder concrete, poured inside of a plank form set around the column, a coat of liquid cement being first applied to the metal with a brush. The plank form should be set at least 2 ins. outside of the metal. Concrete poured in this way, cannot be dislodged by streams of water, and it also greatly strengthens the column.

Fig. 8 shows the method of protecting steel columns employed by the Roebling Construction Company. The column is first furred by vertical rods held in place by clamps, and then by band iron laced to the rods, stiffened wire lath is then bent around and laced to the furring. The space between the lathing and the column is then filled with a moderately dry mixture of cinder concrete. The lathing in this construction is used principally as a form to confine the concrete, in place of a temporary wooden form, but it also serves to prevent the

concrete from being washed away during a fire. Where wooden forms are used the concrete can be given much greater strength, so that lathing is unnecessary, although it forms an additional safeguard.

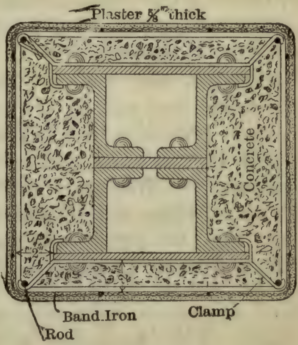

FIG. 8.

In many buildings having reinforced concrete floors, the columns are protected simply by metal lath and plaster. When but a single covering is provided, as in Fig. 17, Chapter XXII, the protection cannot be considered as fire-proof, but when two coverings are provided, as in Fig 9, it is probably all that is necessary for cast-iron columns. The greatest defect in lath and plaster for fire-proofing is that the plaster is liable to be dislodged by the force of the water from the fireman's hose.

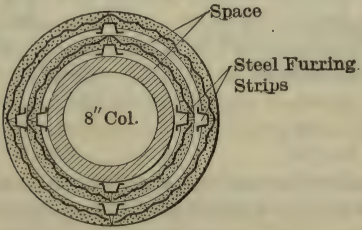

FIG. 9.

When there are two coverings, however, this danger is reduced to a minimum.

Plaster Column Covering—Plaster blocks have been used in buildings as a column covering, but their use is not to be recommended. While it is true that their non-conductivity is in their favor, they are difficult to secure firmly, the plaster tends to promote corrosion in the metal; they are easily washed away by hose streams, and they are subject to greater damage than other materials. In unimportant work their cheapness may, at times, justify their use.

Probably the most defective part of the coverings of columns,

whatever the material used, is about the connections with the beams and girders. Concrete undoubtedly is better adapted for covering this portion of the column than any other material, as, being plastic, it can be made to fit into any space and around any form of connection.

Recesses for Pipes.—"As a matter of economy, both in original cost and in the matter of space, it has been the common practice to run water-, waste-, and vent-pipes immediately alongside the steel columns, and inside the fireproofiing inclosure." * This is undoubtedly bad construction, and in the better types of fire-proof buildings, the pipe space is separated from the columns by the fire-proofing.

Figs. 10 and 11 show the method of running the pipes in

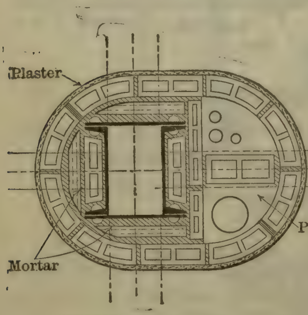

Fig. 10.

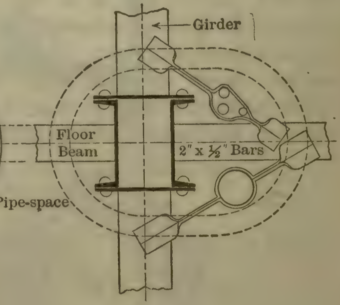

Fig. 11.

some of the latest fire-proof buildings, and is probably as satisfactory as any method where the pipes are to be run beside the columns.

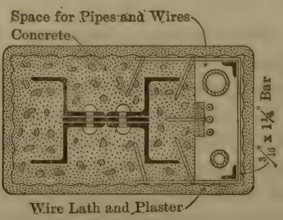

Fig. 12.

Fig. 12 shows a somewhat similar method where concrete and metal lath and plaster are employed for the fire-proofing.

* Freitag.

Fire-proof Floors.*

Fire-proof Floors.—In the study of fire-proofing materials by far the largest attention has been given to floor-construction, and a very large number of types have been developed, of which the characteristic and leading types will here be considered.

Requirements for a Fire-proof Floor.—It goes without saying that a fire-proof floor shall be made of incombustible materials, nor does it seem necessary to mention that it shall resist as much as possible the transmission of heat, so as to afford thorough protection to the metal encased by it or forming an essential part of it. The materials used should not disintegrate or otherwise fail when exposed to heat or flame. They should also resist the action of water that may be used in the extinguishment of fire. The floor-construction should be essentially water-tight, so as to prevent damage by water in a floor below. It should be capable of safely carrying its load at all times.

The New York Building Law, after describing certain acceptable forms of fire-proof floors, provides for a fire test on other types as a precedent condition for their approval. More than fifty tests have been made under the auspices of the New York authorities and these with a few made by other city authorities comprise practically all made in this country. The British Fire-prevention Committee of London have also made a number of such tests.†

The following standard test, recommended by the "Committee on Fireproofing" of the American Society for Testing Materials, is essentially the same as required by the New York Building Code and as used by the British Fire-prevention Committee.

Proposed Standard Test for Fire-proof Floor Construction.

The test structure may be located at any place convenient to the applicant, where all the necessary facilities for properly conducting the test are provided.

* For a more complete discussion of the subject of fire-proof floors, the reader is referred to Chap. IX, of Part I of Kidder's "Building Construction and Superintendence" and J. K. Freitag's "The Fireproofing of Steel Buildings."

† For a list of these tests made in the United States and in London, see Proceedings of Am. Soc. T. M., Vol. VI, p. 128.

The test structure may be constructed of walls of any material not less than 12 inches thick, properly buttressed on all sides.

The floor construction to be tested shall form the roof of the test structure.

At a height of not less than 2 feet 6 inches, nor more than 3 feet above the ground level, a metal grate, properly supported, shall be provided, covering the whole inside area of the building.

In the walls below this grate level, draught openings shall be provided, as many as possible, furnishing openings with an aggregate area of not less than 1 square foot for every 10 square feet of grate surface. Means for temporarily closing these openings should be provided.

In the wall, immediately above the grate level, a firing door, 3 feet 6 inches by 5 feet high, must be provided in the side of the building at right angles to the floor beams. A second door must be added when the span of the floor slab under test exceeds 10 feet.

Flues should be supplied at each of the corners, and oftener in case of a test structure exceeding 250 square feet of grate surface, with sufficient opening to insure a proper draught, securely supported and disposed at the sides of the structure in such manner as not to rest on the floor under test. In no case should a flue area be less than 180 square inches.

The horizontal dimensions of the test structure will depend upon the number and the span of the systems under consideration. The clear span of the floor beams is to be 14 feet. The distance between floor beams, or span of slab, may be varied according to the design of the system to be tested, and should be as near as possible to usual practice. The underside of the construction under test must not be less than 9 feet 6 inches nor more than 10 feet above the grate level.

The construction to be tested should be designed for a working load of one hundred and fifty pounds per square foot, and no more. This load to be uniformly distributed without arching effect, and to be carried on the floor during the fire test.

The floor may be tested as soon after construction as desired, but within forty days. Artificial drying will be allowed if desired.

The floor is to be subjected to the continuous heat of a wood fire, averaging not less than 1700° F. for four hours.

The heat obtained shall be measured by means of standard pyrometers, under the direction of an experienced person. The type of pyrometer is immaterial so long as its accuracy is secured by proper standardization. The heat should be measured at not less than two points when the main floor span is not more than 10 feet, and one additional point when it exceeds 10 feet. Temperature readings at each point are to be taken every three minutes. The heat determination shall be made at points directly beneath the floor so as to secure a fair average.

At the end of the heat test a stream of water shall be directed against the underside of the floor, discharged through a one and one-eighth inch nozzle, under sixty pounds nozzle pressure, for ten minutes.

After the floor has sufficiently cooled, the load on the same shall be increased to six hundred pounds per square foot, uniformly distributed.

The test shall not be regarded as successful unless the following conditions are met: No fire or smoke shall pass through the floor during the fire test; the floor must safely sustain the loads prescribed; the permanent deflection must not exceed one-eighth inch for each foot of span in either slab or beam.

Types of Floor Constructions.—In considering the several systems of floor-construction, they are for convenience divided into the following types or groups:

1. Brick Arches.
2. Terra-cotta Floors.
    - *a*. Segmental.
    - *b*. Flat Side-construction.
    - *c*. Flat End-construction.
    - *d*. Serrated.
    - *e*. Reinforced Tile Arches.
    - *f*. Guastavino.
3. Concrete Floors.
    - *a*. Segmental.
    - *b*. Flat Reinforced Floors.
    - *c*. Sectional Systems.
4. Composition Systems.

Brick Arches.—The first attempt at fire-proof floor construction between wrought-iron beams was by using brick arches sprung between the beams and resting on the bottom flange, as illustrated by Fig. 13.

The bricks should be hard, well-burned brick or hollow

brick of good shape laid to a line on centers without mortar with their lower edges touching, and all the joints filled in with cement grout. The bricks of one line should break joints with those of the next adjoining, and in case of more than one

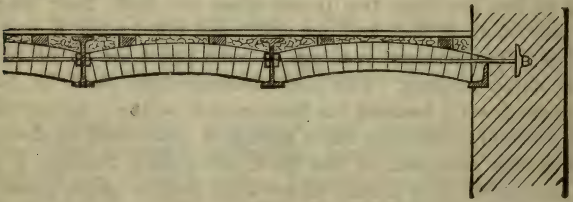

FIG. 13.

row, the joints of one row should also break joint with that of the one above or below.

The arches need not be over 4 inches thick for spans between 6 and 8 feet, provided the haunches are filled with a good cement and gravel concrete put in rather wet. The rise of the arch should be about $\frac{1}{8}$ of the span, or $1\frac{1}{2}$ inch to the foot, and the most desirable span is between 4 and 6 feet.

The building laws of many cities provide that when the spans exceed 5 feet the arches shall be increased in thickness, generally to 8 inches.

The haunches should be filled with concrete level with the top of the arch.

In first-class fire-proof construction the bottom flanges of the beams should be protected by terra-cotta skew-backs,

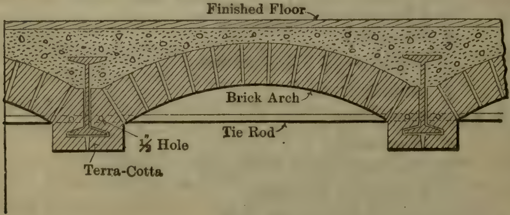

FIG. 14.

as in Fig. 14, which shows the construction used in the main floors of the Government Printing Office at Washington.*

* A description of the structural features of this building may be found n the *Engineering Record* for Dec. 6, 1902.

A 4-inch brick arch of 6-foot span, well grouted and leveled off with Portland cement concrete, should safely carry 300 or 400 lbs. to the square foot. Experiments have shown that brick arches will stand very severe pounding and a great amount of deflection without failure.

The weight of a floor, such as shown in Fig. 1, will usually vary from 70 to 75 lbs. per square foot, depending upon the amount of concrete required for levelling.

Tie-rods, as described on p. 802, should always be provided.

The brick arch is the strongest type of arch for the span it occupies, with the exception perhaps of the stone concrete arch. It is perhaps also the most expensive. Its weight makes a heavier framework necessary than for other types. It is not well adapted for other than warehouse buildings on account of appearance.

Terra-cotta Arches.—Terra-cotta as a fire-proof material and the relative merit of dense, porous, and semi-porous tile have been discussed on p. 732. For floor-construction the semi-porous tile is probably best as compromising on the advantages and disadvantages of the dense and porous tile, particularly as to strength and fire resistance.

As indicated on p. 747 six different types of terra-cotta floor construction, including a larger number of systems, will be discussed. For these a great variety of shapes and sizes of blocks, of the dense, porous or semi-porous material, are manufactured in this country.

The largest company devoted to the manufacture and erection of hollow-tile fire-proofing material is the National Fire-proofing Co., of New York and Chicago. Other large companies are Henry Maurer & Sons, New York; the Haydenville Co., Haydenville, Ohio; Delaware Fire-proofing Co., Delaware, Ohio; and the Illinois Terra-Cotta Lumber Co., of Chicago. Any one of these companies can make any form of block desired, except such as are covered by letters patent, and as a rule in either dense, porous or semi-porous material.

Advantages of Terra-cotta Arches.—Many architects prefer the use of terra-cotta arches in buildings because there is less disturbance to the mechanics of other branches. During the placing of concrete arches there is such a continual dripping as to interfere seriously with other work. The work of installing tile arches is generally more rapid than for other types and it is not necessary to wait for them to dry out. The quality

of terra-cotta can be readily judged from its appearance, not only before being placed but also after it is set, thus not requiring the constant supervision necessary for materials that are mixed as they are put in place. This type of construction, generally speaking, assists to a greater extent than other types in the stiffening of the building against lateral forces, such as wind.

Disadvantages of Tile Arches.—The principal disadvantage of tile arches for floor construction is the difficulty of adapting any system to the filling of irregular-shaped spaces. The arches must be set between I-beams or channels, and to get the best effect the supporting beams must be parallel or nearly so. It is also more expensive to adapt the tile systems to a panel of varying width than with the concrete systems. Tile arches, especially of the end-constructions, are weakened more by holes for pipes than a monolithic floor. As there is no bond between the rows of tiles in the end-construction arch, if a single tile in a row is cut out or omitted there is nothing to hold up the remaining tile in the row except the adhesion of the mortar in the side joints. In this respect side-method arches have an advantage over the end-construction. Where it is necessary to use considerable concrete filling over the arch the weight of the floor construction will usually greatly exceed that of the concrete systems, and this additional weight also means additional expense.

The floor blocks are liable to breakage and chipped blocks in the floor are not unusual. This is perhaps more liable to occur when, as in some localities, the arches are set by bricklayers. When the manufacturers can use their own men, better work can be expected.

Inspection.—Flat arches of hollow tile require close inspection during erection to see that broken or imperfect tile are not used; that the ribs in end-construction tile abut opposite each other; that all joints are properly mortared, and that all of the steel work is properly protected. Very much poor workmanship has been allowed to pass rather than to avoid delay, and also because it cannot be discovered until the centering is removed. A tile arch will generally look better on top than on the bottom. The great carelessness which may obtain in the setting of tile arches was well shown by an article in the *Engineering News* of April 14, 1898.

Setting of Tile Arches.—Tile arches are always set on

wooden centres suspended by bolts hooked over the tops of the I-beams. For all spans of 5 feet and over the centres should be slightly cambered. Before any floor arches are set all girders projecting below floor beams should be completely covered on bottom and sides, independent of the floor construction. To protect the steel from rust it should have a good coat of Portland-cement mortar before applying the tile. After the centres are in place the beam tile should first be placed under bottom of beams and mortar slushed on the sides. Then cover the entire side of the skew-backs which rest against the floor beams with just enough mortar to give a perfect bearing, and shove it up against the beam. Then follow up with intermediate blocks, covering the ribs on one end and one side with a full bed of mortar, and shove in place.

The key should have mortar on both sides and one end (if side-method key is used); it should fit snug, but not tight. "Under no conditions should a key be rammed in place. It is better to use a smaller key and fill out the space left with either a solid slab of tile, or if the opening is too small by a piece of slate."—(E. A. Hoeppner.)

"In setting tile arches it is very common to build the arches in string courses, first fitting all the skews, then all the intermediates, and finally all the keys. This is bad practice, as it loads the centre, both planks and stringers, to excess, causing too great a deflection. In the end-construction the arches should be built one by one, each being complete before the next is started. In side-construction, where joints are broken longitudinally, the arches should be keyed up or completed at the first point where the intermediates meet the lines of the key, thus completing the successive arches as rapidly as possible." —(Freitag.)

All joints in the arches should be filled with mortar, especially at the top.

Wetting the Tile.—In warm weather all hollow tile, whether dense or porous, should be well wet or water-soaked before laying. In freezing weather they must be kept dry.

Mortar for Setting.—"Mortar for setting porous hollow tile should never be made of cement and sand alone, as such mortar is too 'short' and rolls off the tile and does not insure a full joint."—(E. A. Hoeppner.)

One part Portland cement added to three parts rich cold lime mortar makes a good mixture for either dense or porous tile.

A better mortar is made by mixing the cement and sand and adding enough cold lime putty to make it work smooth. The mortar should be thoroughly worked. Hot lime mortar should never be used.

In dry weather the centres can be removed in 36 hours after the tile are in place, but it is much better to allow 48 hours and even longer in cold or wet weather.

Filling above Tile Arches.—The strength of all tile arches is greatly increased by wetting the tops of the arches and covering with a rich cinder concrete (mixed with Portland cement), well tamped and brought level with the tops of the steel beams.

If the floors are to be finished in wood, nailing-strips are required for securing the flooring.

These nailing-strips are usually of a dovetail shape about 2½ inches wide at the top, 3½ inches at the bottom and 1¾ to 2 inches thick. It is preferable to lay them at right angles to the steel beams, so that they may be secured to the top flange by a metal clip, as in Fig. 15. Before the nailing-strips are laid all piping and wiring which must go above or through the tile arches should be put in place. After the nailing-strips are in place the tops of the steel beams should be covered with a thin coat of Portland cement and sand grout, applied with a brush. The spaces between the nailing-strips should be filled with a 1-to-8 or 10-cinder concrete, finished about ¼ inch below the top of the strips.

Terra-cotta Filling-blocks.—In cases where the tops of the tile arches are 2 inches or more below the tops of the steel beams, hollow terra-cotta blocks are sometimes used for filling to the top of the beams, as in Fig. 28. These blocks are lighter than good concrete, but they do not strengthen the arch unless they are set in cement mortar and all the tiles and the tops of the arches well wet just before laying the filling-blocks.

Cement Floors.—If the floors are to be finished with cement, the cement and concrete should be at least 2½ inches and preferably 3 inches thick above the steel beams, and should be blocked out in sections of not over 6 feet square, with joints extending through the concrete. When practicable the joints in one direction should be over the beams.

Weather Protection.—Terra-cotta arches should always be protected against rain or snow, especially in freezing weather, as both the blocks and the mortar in the joints are injured by

freezing. Porous terra-cotta especially may be utterly ruined by freezing when soaked with water.

Protection from Stains in Ceiling.—"If plastered ceilings are to be used, the terra-cotta work should be protected against the smoke or soot from the hoisting-engines. Stains are also quite liable to occur from the effects of iron in the clay, or from the cinders in the concrete over the arches, if the floor is allowed to become wet."—(Freitag.)

To prevent these stains several hydraulic paints have been used, some of which have proved very effective.*

Segmental Terra-cotta Arches.—"This form of arch is the strongest and cheapest. It is particularly adapted to warehouses, lofts, factories, sidewalks, or wherever great strength is required and a flat ceiling is not necessary.

"When a light, strong arch is required in deep beams and a flat ceiling is also demanded, this result can be obtained by using a metal lath ceiling suspended below the beams." †

These arches are usually formed by either 6″ or 8″ hollow tile, set on the side-construction principle and bonded endways like a brick vault. They can be used for spans up to 20 feet, but it is better to limit the span to about 16 feet.

"End-construction blocks may be used, but they are unsatisfactory, unless the arches are of uniform span and rise throughout. The rise of the side-construction arch can be varied by increasing the thickness of the upper or lower part of the mortar joint, but this cannot be done with the end-construction method." †

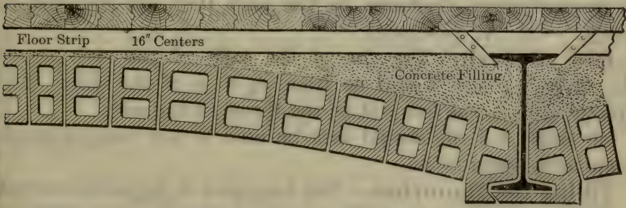

Fig. 15.

Figs. 15, 16, and 17 show typical forms of segment arches.

The weight of the arch tile will run about 26 lbs. per square foot for 6-inch tile and 32 lbs. for 8-inch tile. To these weights

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* See Antihydrine, page 403, Building Construction, Part I.

† Bevier, National F. P. Co., N. Y.

should be added the weight of concrete filling, flooring, plaster, etc.

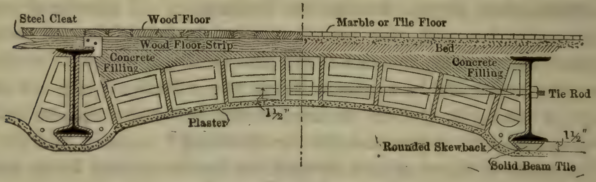

FIG. 16.

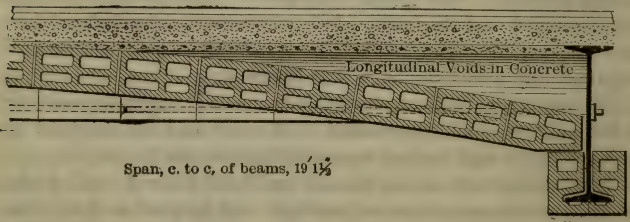

FIG. 17.

Thickness of Webs.—"For general use the webs of segment tile should be ½ inch thick for semi-porous tile and ¾ inch for porous tile. The skew-back should be at least ¾ inch thick for the first-named material and 1 inch for the second. For printing establishments or any other building where a large amount of vibration occurs the webs of all tile must be designed in proportionate thickness to the load they are required to carry."—(E. A. Hoeppner.)*

Rise of Arch.—The rise of the soffit of the arch above the springing line should be from $\frac{1}{10}$ to $\frac{1}{8}$ of the span. The greater the rise the less will be the thrust of the arch.

No single cell tile should ever be used in any form of terra-cotta arch construction.

Filling the Haunches.—The haunches of segmental arches should be filled with good cement concrete levelled up to a point not less than 1 inch above the crown of the arch. For short spans cinder concrete filling may be used, but for wide spans it is better to use gravel concrete, as the strength of the

* These thicknesses apply to Chicago practice more particularly, where a stronger tile is produced than in the East. In New York webs are generally ⅝″ for semi-porous and 1″ for porous tile.

arch at the haunches depends largely upon the strength of the concrete filling. Voids are sometimes formed in the haunches, stiff pasteboard being used to form the core.

Tie-rods.—The thrust of segmental arches is very considerable, so that it is important to provide plenty of tie-rods between the beams. A formula for determining the stress in the tie-rods and the diameter of the same is given on page 804.

To be most effective the tie-rods should be spaced within the lower third of the beam, or preferably at the centre of the skew.

Placing the tie-rods within the lower third of the beam, however, will cause them to project below the soffit of the arch, giving an unsightly appearance to the ceiling and rendering

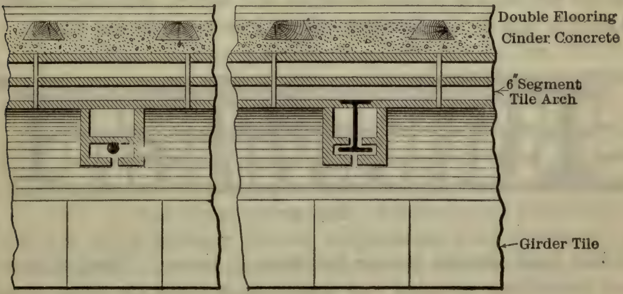

FIG. 18.—Longitudinal sections.

them difficult to protect. Occasionally the tie-rods are encased with special tile, as in Fig. 18.

Strength of Segmental Arches.—The safe loads per square foot on 6- and 8-inch segmental arches, side construction, of semi-porous tile, having a rise of one-eighth of the span, with webs and shells of $\frac{5}{8}''$ thickness, with a factor of safety of 7, as obtained from table of the National Fire-proofing Co., are as follows:

Span, Feet.	6-inch Arch, Pounds.	8-inch Arch, Pounds.	Span, Feet.	6-inch Arch, Pounds.	8-inch Arch, Pounds.
4	1103	1318	11	402	480
5	878	1049	12	370	442
6	735	883	13	340	407
7	630	735	14	317	379
8	554	662	15	296	353
9	490	585	16	278	331
10	443	529			

These loads include the weight of the construction, so that to get the safe live load all the dead load of arch block, concrete fill, plastering, flooring, etc., must be deducted.

Side Construction Terra-cotta Arches.—By this term is understood the flat tile arches in which the voids in the blocks run parallel with the beams, as shown in Fig. 19.

On account of its greater strength at nearly the same cost the end-construction arch has largely replaced this form of floor arch, the former now being used to about three times the extent of the latter.

One advantage of this arch over the end-construction, however, is the breaking of joints that is effected in the setting of the blocks, by means of which the failure of a single block does not impair the strength of the arch beyond that block.

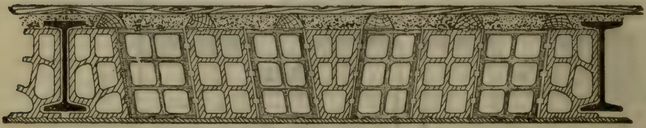

Fig. 19.

The webs should not be less than $\frac{5}{8}''$ thick. "Radial joints are sometimes specified but should be avoided, as they incur needless expense in manufacture and endless confusion and delay in setting without any compensating advantage." *

In the skew-backs a web should always be provided across the block at the lower flange of the beam, as at this point comes the greatest pressure in this block. Arches have collapsed for failure to provide this web.

The depth of the arch must be proportioned to the span between beams and to the load to be carried. For ordinary loads a safe rule is to make the depth of the block $1\frac{1}{4}$ inches for each foot of span, plus the amount necessary for protection below the beams.

Safe loads for semi-porous tile arches, side construction, with webs $\frac{5}{8}''$ thick and a factor of safety of 7, as given by the National Fire-proofing Co., are shown in table on page 757.

These loads represent the gross loads, so that for the safe live loads the weight of the construction, including arch blocks, fill, flooring, plastering, etc., must be deducted. For blocks

* Bevier, National F. P. Co., N. Y.

Depth of Arch..	6 Ins.	7 Ins.	8 Ins.	9 Ins.	10 Ins.	12 Ins.
Weight of Arch per Sq. Ft...	24 lbs.	26 lbs.	27 lbs.	29 lbs.	34 lbs.	37 lbs.
Span of Arch in Feet.	Strength of Arch in Pounds per Square Foot.					
4′ 0″	197	230	263	296	438	525
4′ 6″	156	182	208	233	346	415
5′ 0″	....	148	168	189	281	336
5′ 6″	....	....	139	156	232	278
6′ 0″	....	....	....	131	195	234
6′ 6″	....	....	....	....	166	199
7′ 0″	....	....	....	....	....	172

with thicker webs the loads may be increased proportionately. Where no loads are given in the table, the spans are considered excessive for the depth of block specified.

The weights of arch given in the table are for the lightest blocks. If thicker webs are used weight of block must be taken proportionately greater.

End-construction Flat Arches.—In this construction the sides and voids of the individual blocks run at right angles to the beams, so that the pressure on the blocks is endways of the tile. It has been conclusively demonstrated that hollow tile are much stronger in end compression than transversely, consequently the end-construction arch has to a large extent superseded the side construction.

"The objection urged against this construction is that it is wasteful of mortar and difficult to get the edges of the blocks properly bedded. They do require slightly more mortar, but the second objection is not serious, for, if the blocks are cut to a proper bevel, the tighter they are set the stronger the arch." *

The individual blocks in the end-construction are commonly made rectangular in shape and advancing by 1 inch from 6 to 15 inches in depth. The length and width of the blocks may also be varied, but the standard size is 12 inches for both dimensions. The number of partitions or webs in the blocks varies with the size of the block and also with the strength desired. The 6-, 7-, and 8-inch blocks usually have two vertical and one horizontal partitions, or one vertical and one horizontal for blocks 8 inches wide. The 10- and 12-inch arches may have either one or two horizontal partitions. Arch blocks over 12 inches deep should always have at least two horizontal

* Bevier.

partitions. In the strongest blocks the voids are about 3 inches square.

"The arch blocks must be set end to end in straight courses from beam to beam, and cannot be set breaking joints, as in the side-construction method." * So that if one block fails, the rest of the arch, for the width of that block, is dependent for its strength on the adhesion of the individual blocks to those adjoining.

Thickness of Web.—This should be at least ¾ inch for porous tiling and ½ inch for semi-porous. The thicker the webs the greater will be the strength of the arch, and also its fire resistance.

The end joints are always bevelled, as in Figs. 20 and 21,

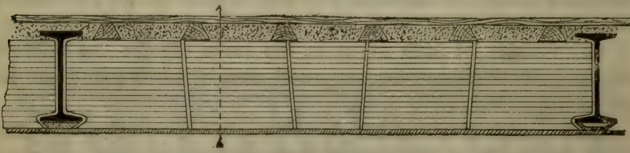

Fig. 20.

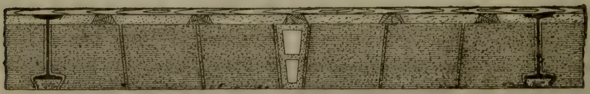

Fig. 21.

the ends being parallel, thus all the intermediate blocks are made with the same die.

Form of Skew-backs.—An end-construction arch may have skew-backs formed of the same blocks, with a notch in the end of the block to fit over the bottom flange of the beam, as in Fig. 22. It is generally considered that the end-construction skew is much stronger than the side-construction skew, but on account of the large amount of mortar lost in the voids and the difficulty of obtaining an even bearing with end-construction skews, and also because of the greater facility with which the side-construction skew-backs can be used, contractors generally prefer to use the latter and this has given rise to the *combination arch*, shown by Fig. 23.

But a more important reason for using side-construction

* Bevier.

skew-backs with end-construction arches, is the better protection against fire they afford to the beam or girder.

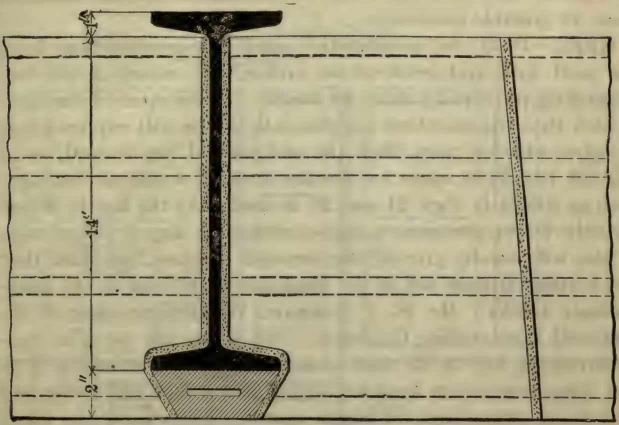

Fig. 22.—End-construction Skew-backs.

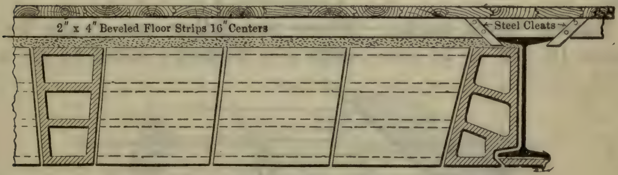

Fig. 23.—Combination Arch.

To develop the necessary strength side-construction skews should have a large sectional area and a sufficient number of partitions, following, approximately, the lines of thrust.

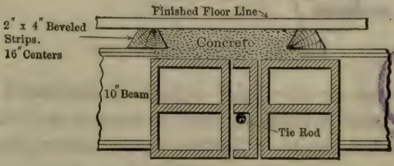

Fig. 24.—Longitudinal Section.

With any form of skew the recess for the beam flange should be wide enough so that when the tiles are set the protection

flange on the skew will not touch the bottom of the beam, but will be at least ¼ inch below it.

A great variety of side-construction skew-backs are made to meet all possible conditions.

Keys.—Both end-construction and side-construction keys are used with end-construction arches, the choice of the key depending principally upon its length. If the span of the arch is such that the standard intermediate blocks will require a key 6 inches wide or more, then the end-method key is used, as in Fig. 20, but if the space for the key is small, a side-method key, such as shown in Figs. 21 and 23, is used. As the key is almost entirely in compression, a side-construction key 6 inches wide or less will usually give all the strength required, provided that the horizontal webs are in the same line with those in the intermediate blocks. Mr. E. V. Johnson, Western manager of the National Fire-proofing Co., says: "We prefer the use of an end-construction key in all cases where possible. Our custom is to use side-construction keys for spaces of 6 inches and under and end-construction keys for larger spaces. When using the latter keys we insert a ⅝-inch fire-clay slab between the ends of the tile."

Raised Skew-backs.—Where flat arches are sprung between 18-, 20- or 24-inch beams it is necessary either to use a

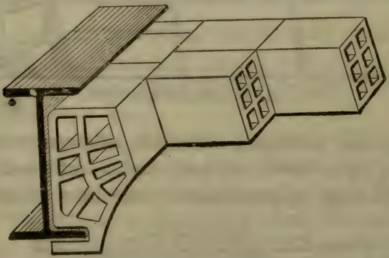

Fig. 25.—Raised Skew-back.

raised skew-back or else have a large space above the top of the tile arches which must be filled in some way. Raised skew-backs are preferable to a hollow space above the tiles and cheaper than concrete filling. They are often used for roof arches, because it is seldom necessary to make the arches as deep as the beams, while the top must be about on a level with the beams.

Raised skew-backs are almost always made on the side-construction method, Figs. 25, 26, and 27 showing typical forms for end-construction arches.

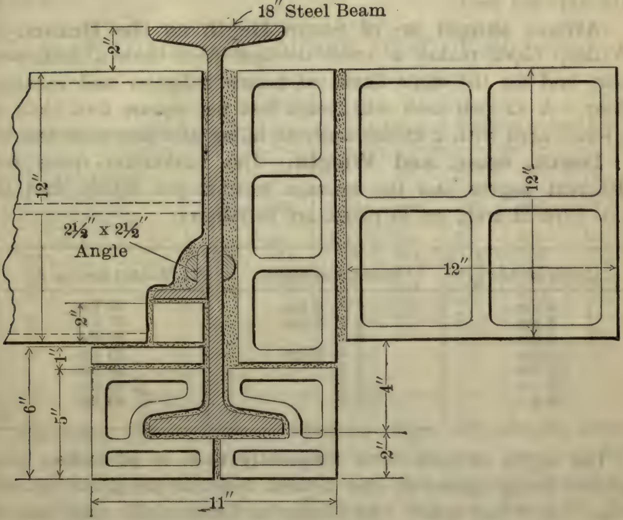

FIG. 26.

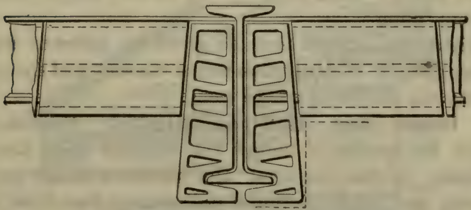

FIG. 27.

Flat vs. Panelled Ceilings..—In connection with the raising of the arches above the bottom of the beams or girders, Mr. Freitag calls attention to the advantages of flat ceilings, as follows:

"Flat, unbroken ceilings are always to be preferred to any type of terra-cotta arch which may require a panelled effect due to the projection of the girders or beams below the main ceiling

line." A perfectly flat ceiling reflects more light and gives a better-lighted room and also deflects heat. Panelling forms pockets for the retention of heat and flame and greatly increases the exposed area.

Arches should be of Same Depth as the Beams.—A deep block makes a much stronger floor than a shallower one, and for the same depth of beams a lighter and cheaper floor. A 12-inch arch will weigh less per square foot than a 10-inch arch with 2 inches concrete filling and also costs less.

Depth, Span, and Weight.—The maximum spans for different depths and the average weights per square foot of this type of arch, set in place, are as follows:

Depth of Arch.	Maximum Span.	Weight per sq. ft.
6 ins.	4 ft. 6 ins.	29 lbs.
8 ins.	5 ft. 6 ins.	31 lbs.
9 ins.	6 ft.	32 lbs.
10 ins.	6 ft. 6 ins.	33 lbs.
12 ins.	8 ft.	39 lbs.
15 ins.	9 ft.	46 lbs.
16 ins.	10 ft.	50 lbs.

The depth of arch most frequently used is 10 inches, the girders being spaced to use 10-inch I-beams for joists spaced from 5 to 6 feet apart. As a rule the depth of the arch should be about equal to the depth of the beam, as it is just about as cheap and much better construction to use deeper tiling and less concrete filling.

The weights per square foot, as given by different manufacturers, vary greatly, no doubt due to the character of the material used and to the thickness of the webs.

Safe Loads for End-construction Arches. — The strength of flat arches of hollow tile depends upon the crushing resistance of the material, the sectional area, per lineal foot of arch, and upon the depth and span. For these reasons it is impossible to give a table for strength which will apply to all arches. The table on the following page is condensed from two tables prepared by Mr. H. L. Hinton, who has gone very elaborately into the strength of tile arches, in the handbook prepared by him for the National Fire-proofing Co.

The values given for end-construction arches are based on arch-blocks of the cross-sectional areas (per foot) given at the head of the table, and are intended to have a factor of safety of 7 with the weight of the tile only deducted.

Mr. Hinton says: "The safe loads as they stand in the table afford a safe general statement of safe loads for *all sections*, since they represent specifically a light section in the case of each arch."

SAFE LOADS PER SQUARE FOOT OF FLOOR.

END-CONSTRUCTION FLAT ARCHES.

(H. L. HINTON.)

Semi-porous material of sectional area per lineal foot as given in second line.

Depth of Arch.	6″	7″	8″	9″	10″	12″	15″
Areas, Sq. Ins.	310	340	370	400	430	490	580
Spans.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
4′ 6″	196	254	319	391	470	648	968
5′	155	202	254	312	376	519	777
5′ 6″	......	163	206	254	306	424	636
6′	......	......	170	209	253	352	529
6′ 6″	......	......	141	175	212	295	446
7′	......	......	......	147	179	251	380
7′ 6″	......	......	......	......	153	215	326
8′	......	......	......	......	......	185	282

Patented End-method Arches.—Figs. 28 and 29 show two variations of a type of arch invented and patented by Mr. E. V. Johnson when manager of the Pioneer Company, of Chicago. The right to manufacture and use this arch, in certain territory, was granted to the Pioneer Company; also

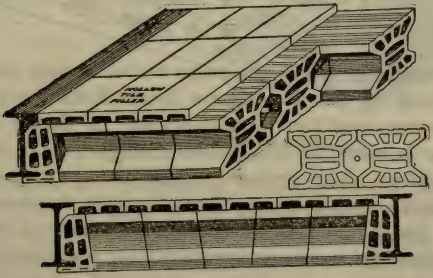

FIG. 28.

to Henry Maurer & Son, of New York, and to the Haydenville (Ohio) Company. The original shape of the arch tile is illustrated by Fig. 29, and this shape is still used by the Pioneer Company. Henry Maurer & Son have modified the shape to that shown by Fig. 28, considering that this shape gives a stronger

(and also a slightly heavier) arch than the original shape. The advantages of this arch are reduced weight, with equal strength and a clear space of 5 inches between the tile, which avoids cutting of the blocks for the tie-rods.

This arch can be adapted to any span up to 10 feet by using a suitable depth of block.

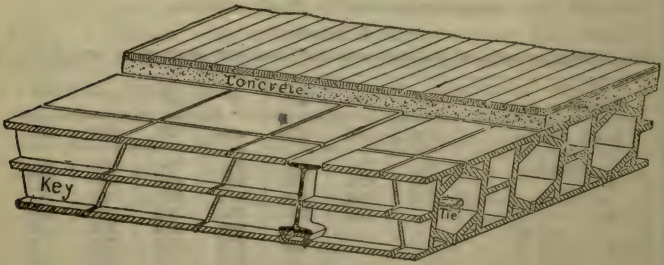

FIG. 29.

The limit of span, weight per square foot and safe load of the "Excelsior" arch is given by Maurer & Son as follows:

Depth of Arch.	Limits of Span.	Weight per sq. ft.	Safe Load per sq. ft.
8 ins.	5 ft. to 6 ft.	27 lbs.	300 lbs.
9 ins.	6 ft. to 7 ft.	29 lbs.	350 lbs.
10 ins.	7 ft. to 8 ft.	33 lbs.	300 lbs.
12 ins.	8 ft. to 9 ft.	38 lbs.	350 lbs.

The Pioneer Company have made arches as deep as 20 inches and weighing 56 lbs. per square foot. Both companies use semi-porous material for the arch-blocks. It should be noticed that the arch made by the Pioneer Company has an end-construction skew, while Maurer & Son use a side-construction skew. The Pioneer Company formerly used the side-construction skew, but found that when arches of this type were tested to destruction the skew-backs were almost invariably the parts which failed, hence their adoption of the end-construction skew. Messrs. Maurer & Son, however, have tested the Excelsior arch, with spans of 8 and 10 feet, with loads of over 1000 lbs. per square foot, without failure, with skew-backs as shown by them.

This arch has been very extensively used in both Eastern and Western cities and is undoubtedly a very good type.

Reinforced Tile Arches.—In order to obtain a wide span flat arch or to obtain a reduced depth of arch block for the shorter spans, the manufacturer of terra-cotta have applied to their floor construction the principle of reinforcement with metal that is the basis of reinforced concrete construction.

Compared with reinforced concrete, even when cinders are used for the aggregate, their greater depth and hollow construction secure for them greater strength for the same weight of construction per square foot. On the other hand, however, they are undoubtedly more expensive than cinder concrete construction, because of the material used and the increased height of the building due to thicker floors.

The "Herculean" Arch.*—This floor is built of semi-porous terra-cotta blocks 12 inches by 12 inches on top and

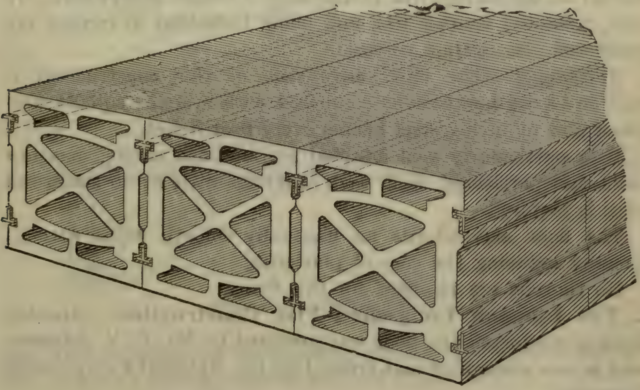

FIG. 30.—The "Herculean" Arch.

varying from 6 to 12 inches in depth, according to the span and load. In the sides of the blocks are grooves to receive $1\frac{1}{2}\times1\frac{1}{2}\times3/16$-inch T-bars. The blocks are laid end to end the entire length of the span, with a bearing of 4 to 6 inches on the walls or girders, presenting two continuous grooves, which are filled with cement, and into which the T-bars are then inserted. The T's must, of course, extend the full length of the span. The grooves in the next course are then filled with cement and the blocks pushed into place, thus thoroughly covering the steel with cement.

* Patented and manufactured by Henry Maurer & Sons, 1898 and 1900.

All joints between the blocks are filled with cement and the blocks are laid to break joint endways, as in Fig. 30.

Span.—This floor is adapted to spans up to 25 feet and has been extensively used for spans varying from 19 to 23 feet.

Weight.—The weight of the terra-cotta blocks and steel tees per square foot is given at 26 lbs. for blocks 6 inches deep, 33 lbs. for 8-inch blocks, 42 lbs. for 10-inch blocks, and 51 lbs. for 12-inch blocks.

Strength.—The manufacturers estimate the safe load for this construction as follows:

For 12-inch arch, 20-foot span, 400 lbs. per square foot.
For 10-inch arch, 16-foot span, 400 lbs. per square foot.
For 8-inch arch, 12-foot span, 150 lbs. per square foot.

Fire-proofing Qualities.—The steel tension members, being buried in the terra-cotta blocks over 2 inches everywhere, are well protected against fire, and being imbedded in cement are protected also against corrosion.

Advantages.—The chief advantage of this construction is said to be its low cost as compared with systems equally fire-proof and requiring steel beams every 6 or 8 feet.

It is particularly well adapted to buildings having masonry walls and partitions, as in such buildings little or no structural steel is required.

The floor also affords an unusually smooth under surface, thereby reducing the cost of plastering.

No tie-rods are required for this floor.

The Johnson Long-span Flat Construction.—Another leading reinforced tile floor was invented by Mr. E. V. Johnson, and is now controlled and erected by the National Fire-proofing Company.

The general construction of this floor is as follows:

A temporary flat centring is first erected and over this is spread a layer of rich Portland cement mortar about $\frac{3}{4}$ inch thick. On top of this mortar is laid a woven fabric containing steel rods varying from $\frac{1}{4}$ inch to $\frac{1}{2}$ inch in diameter, according to the span, and spaced from 2 inches to 8 inches centre to centre. Another layer of the same mortar is then spread on top and hollow tiles from 3 to 12 inches in depth, according to the span, are then set in the mortar and laid with "break joint," so as to form continuous rows from one support to the other, the same as in end-construction flat arches, except that in the Johnson construction the ends of the tile are square to the beds.

A layer of concrete about 2 inches thick is also usually spread on top of the tile.

Fig. 31 shows the general method of construction of this system, but without the rods, which are inserted in place as the fabric is used. For short spans the fabric can be used without the rods.

As already stated, this system differs from the flat concrete systems only in the substitution of hollow tile for the concrete in the upper portion of the slab, its strength depending upon the reinforcement and the adhesion of the cement mortar to the steel and tile. As the tile are covered both on the bottom and

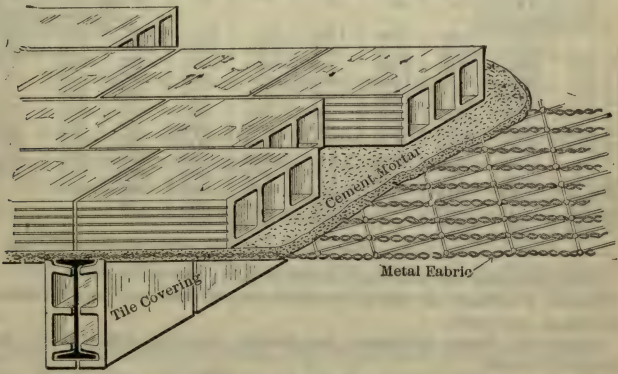

Fig. 31.—Johnson Arch.

top with concrete the fire-proofing quality is also measured by the resistance of the concrete and not of the tile.

Many tests, however, have shown that the adhesion of the mortar is perfect and that it will stand a high temperature without injury.

Span.—This construction can be used for any span up to 25 feet, the most advantageous span being about 16 feet.

Weight.—The weight per square foot of this floor, including the fabric and the cement on the bottom and in the joints, but *not on top of the tile,* is as follows:

For depth of tile of 12 inches, 10 inches, 9 inches, 8 inches, 7 inches, 6 inches, 5 inches, 4 inches.

Weight per square foot, lbs., 60, 55, 45, 42, 37, 34, 26, 24

The concrete above the tile should be figured at 12 lbs. per square foot for each inch in thickness.

Strength.—The proprietors of this system give the following for the ultimate strength of the floors with 1 inch of 1 to 3 Portland-cement mortar on top of the tile.

JOHNSON SYSTEM.

With 1″ Portland-cement Floor Surface.

Spans from 10 to 24 Feet.	Ultimate Strength in Pounds per Square Foot.								
	Thickness of tile. 12″	Thickness of tile. 10″	Thickness of tile. 9″	Thickness of tile. 8″	Thickness of tile. 7″	Thickness of tile. 6″	Thickness of tile. 5″	Thickness of tile. 4″	Thickness of tile. 3″
	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
10 feet	3375	2580	2140	1850	1525	1265	1000	775	560
11 feet	2800	2340	1780	1536	1264	1052	832	640	464
12 feet	2350	1800	1480	1280	1064	880	700	540	390
13 feet	2000	1540	1265	1100	910	752	595	460	334
14 feet	1730	1325	1100	950	780	650	510	400	290
15 feet	1500	1160	950	830	680	590	450	348	250
16 feet	1320	1010	840	720	600	500	395	305	220
17 feet	1180	900	740	640	578	440	350	270	194
18 feet	1020	795	664	570	473	392	310	242	174
20 feet	844	645	535	462	381	314	250	194	
22 feet	700	536	445	384	316	263	208		
24 feet	587	450	370	320	266	220			

With this table the following factors of safety should be used:

Factor 4.—Floors of offices, school rooms, hospital and asylum wards, dwellings, and roofs.

Factor 5.—Floors of stores, warehouses, theatres, public halls, and assembly rooms.

Factor 6.—Floors of buildings where vibration of machinery or loads causing a sudden impact, occurs.

A section of this floor, 16 feet square, supported on walls around the four edges, was loaded over its entire area with a total uniformly distributed load of 187,680 lbs. or 733 lbs. to each square foot. The deflection of the floor was as follows: Under a load of 350 lbs. per square foot, $\frac{1}{8}$ inch scant; 733 lbs. per square foot, $\frac{1}{4}$ inch full.

Advantages.—The advantages of this system are the same as noted for all long-span flat systems; the system can be used to special advantage for roofs and for buildings divided by masonry partitions, so that the spans do not exceed 25 feet. For such buildings very little, if any, structural steel will be required.

"New York" Reinforced Terra-cotta Arch.—This arch (Fig. 32) was designed by Mr. P. H. Bevier, of the New York branch of the National Fire-proofing Co., for use "when

a light and cheap but strong floor construction with a flat ceiling is required, and is particularly adapted to wide spans in shallow beams. When light floor construction with deep beams is necessary it can be secured by setting the blocks level

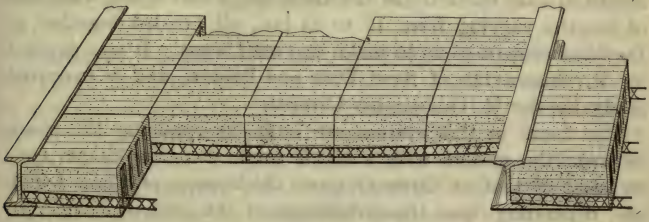

FIG. 32.—"New York" Reinforced Arch (Bevier Patent).

with the tops of the beams and using a flat metal lath ceiling, or by omitting the ceiling a panel effect is obtained."

"When shallow beams are used the blocks are set level and 1 inch below the bottom of the beams. Light cinder concrete or dry cinders is used to level up to the top of the beams."

"The wire truss reinforcement, Fig. 33, used in this system is shipped to the building in reels, and is cut to proper lengths

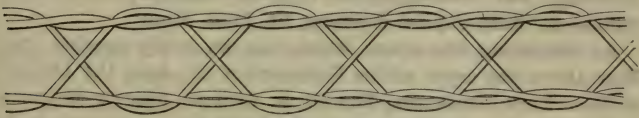

FIG. 33.—Wire Truss Reinforcement.

on the job as required. It is embedded in Portland cement mortar between the blocks, so that it is protected both against rust and fire. The open-work construction of the wire truss enables the mortar to flow freely all about it and the joint can be thoroughly filled between the blocks and the wire perfectly embedded."

"The 6-inch arch for 6-foot span, and 8 inch arch for 7-foot 6-inch span, have been tested by the Bureau of Buildings, of New York, and accepted for live load of 150 lbs. per square foot."

"The 'New York' arch has been used in a number of large buildings in New York. Load tests were made to determine

the ultimate strength of the 6-inch arch on a 6-foot span, and it was found to be 1600 lbs. per square foot."

The Guastavino Tile Arch System.—This is a peculiar method of constructing floors, partitions, staircases, etc., by means of thin tile 1 in. in thickness and about 6 ins. wide and of lengths varying from 12 to 24 ins., all bonded together in Portland cement so as to make one solid mass. It was devised by R. Guastavino of New York and Boston, and is executed solely by the R. Guastavino Company.

The floors in this system are built by spanning the space between the girders with a single arch, vault, or dome, constructed of two or three or more thicknesses of these 1-inch tile, depending upon the dimensions of the arch or vault. In its best application, steel is used in tension only as tie-members, and in place of steel girders tile girders are constructed of the same material; wherever steel is used it is imbedded in the masonry construction.

One of the earliest notable buildings in this system is that of the Boston Public Library, built about fourteen years ago, and some of the later important constructions are the Hall of Fame and Library Building of the University of New York, and the Metropolitan Museum of Art, in New York, Massachusetts Horticultural Building, American Type Foundry Building, and Massachusetts General Hospital, in Boston, Minnesota State Capitol Building, St. Paul, Minn.

As indicating the spans which can be safely applied, the floor above the crypt of the Cathedral of St. John the Divine, in New York, measuring 56×60 ft. with no interior supports and designed to carry a safe load of 400 lbs. per square foot, was constructed on this principle.

Wherever a vaulted ceiling is desired this seems to be the best system of construction yet devised.

Floors built on this principle have been tested under the supervision of the New York Building Department up to 3700 pounds per square foot, with spans of 10 ft.

When used between I-beams the only steel beams required are those spanning from column to column.

Architects contemplating the use of this system of construction are advised to consult the R. Guastavino Company before letting any contracts.

Wherever vaulted ceilings are desired this construction should be as cheap, and generally is cheaper than any other form of

equally fire-proof construction—a particular advantage of the system being that frequently the soffit course of tile is of pressed or glazed material, making a most effective and permanent finish, as in the case of the City Hall Station of the New York Subway, which was constructed for very heavy loads without the use of steel.

Concrete Floors.—Concrete used in fire-proof floors may be either plain or reinforced. Without reinforcement its use is not generally practicable in any thing but very short spans, on account of its expense or great weight. In this chapter it will be considered only as a floor filling between steel beams. The following chapter will be devoted to a discussion of the principles governing its design and use.

Advantages of Reinforced Concrete for Floor Construction.—Although many advantages are claimed for reinforced concrete over the tile systems, the principal advantage is that of economy, taking into account the cost of both the steel framework and the filling between. The other important advantages are less weight per square foot of floor (usually but not always), adaptability to irregular framing and rapidity of construction.

Except in the immediate locality of the tile factories, fire-proof floors of concrete can usually be placed at less expense than those of hollow tile, and when the spans will permit of the use of cinder concrete, the concrete floors will be lighter than those of the tile, when both floors have the same strength. Some of the long-span tile systems, on the other hand, are much lighter than many of the concrete floors that are now being built.

The materials entering into the construction of reinforced concrete floors are readily obtained in almost any locality, no special prepared material is required, except perhaps in a few special forms of reinforcement, and the work can be done almost entirely by unskilled labor. Less capital is required for concrete work than for the tile constructions, and no material need be carried in stock during an idle period, except tools, mixing machines, old centering, etc.

That the above advantages are real is sufficiently proven by the immense amount of reinforced concrete now under construction throughout the world.

Wherever a floor is to have a finished cement surface, reinforced concrete constructions will be considerably cheaper

than any tile system, because in the former construction, the entire concrete is used to give strength, while with the flat-tile arches it merely increases the dead weight.

Disadvantages.—One decided disadvantage connected with concrete floor construction is the interference in a large measure with the progress of other parts of the work. During its installation, there is a constant dripping from the floor, making it sometimes impossible to continue other lines of work. After completion of the floors a long time is required, depending on the weather, for the drying out, before interior finishing can proceed.

Composition of the Concrete.—The materials used for concrete are discussed on p. 733. Only Portland cement should be used in any floor construction.

For most reinforced concrete floors, having a span between the steel beams of 8 feet or less, cinder concrete is generally used for the reason that concrete mixed with cinders is much lighter than that mixed with broken stone or gravel. The usual proportions of cinder concrete are one of cement to two of sand and five or six of cinders.

To make a first-class concrete the cinders must be screened through at least a $\frac{3}{4}$-inch mesh, and only hard coal cinders should be used. Good cinders may sometimes be obtained from power-plants using soft coal, but it must be well screened to free from the ash. Concrete mixed with common ashes, as is sometimes done, has little strength and is totally unreliable.

For all spans exceeding 8 ft., either gravel or broken rock should be used, and these should be mixed with one part cement to two of clean, sharp sand and four of stone or gravel.

The weight of cinder concrete will vary from 80 to 110 lbs. per square foot, depending upon the coarseness of the material, quantity of sand, and the amount of tamping. For ordinary purposes a 1: 2: 5 cinder concrete should be taken at 96 lbs. per cubic foot, or 8 lbs. per square foot per inch in thickness.

Forms of Reinforcement.—While steel in small sections is used almost entirely for the reinforcement, there is a great variety in the shape and character of the metal employed.

Different forms of reinforcement are described and discussed in the next chapter. All of them may be and most of them are being used in floor construction. In addition to those

discussed there, others that are not readily adapted to beam construction are used in floor construction. Such are the metal fabrics described farther on under the different types of construction.

The proper position for reinforcement in a floor construction is that in which it will take the tensional stresses, that is, in floor slabs, near the lower surface. The most logical form would be a rod or bar. A greater number of small rods or bars is preferable to a smaller number of larger ones, because the proportion of the area of adhesion between steel and concrete to the sectional area of steel is greater in the former case. This would apparently be attained in such systems as use wire fabrics. But the disadvantage of the smaller reinforcement is the greater possibility of corrosion and consequent failure. The wire fabrics have the further disadvantage that they are easily displaced in the process of placing of concrete—either getting too low and becoming exposed to fire or corrosion, or getting too high with a corresponding weakening of the floor. Another point that must be remembered in the use of metal fabric, is that the mesh shall be large enough to permit a good bond between the concrete above and below it.

Reinforcement in the form of bars set vertically in the concrete has a tendency to shear through the slab under heavy loads.

The best and most logical reinforcement for fire-proof floors consists of $\frac{1}{2}$ inch to $\frac{3}{4}$ inch round or square rods, either plain or deformed, spaced at varying distances to suit the spans and loads.

Necessity for Longitudinal Bars.—Where wire strands or bars are used for reinforcement it is essential that there be longitudinal as well as transverse bars, for the reason that under heavy concentrated loads, or when a heavy body falls upon the slab the concrete will crack between the carrying bars. This can be readily demonstrated in testing a floor slab without longitudinal wires under a drop test. When the load is uniformly distributed the longitudinal wires are not brought into play, but floor loads are more often concentrated than uniformly distributed.

Segmental Concrete Arches.—For heavy warehouse floors the arched systems are preferable to the flat systems, as the concrete is thus used in its strongest form, and less reinforcement is required. In warehouses, also a ceiling formed of a series of arches is not objectionable.

For spans between floor beams of 5 ft. or less, a 1 to 6 gravel-concrete arch, 3 ins. thick at crown and without any reinforcement should sustain a distributed load of 1500 lbs. per square foot without cracking.

For spans exceeding 5 ft., the celebrated Austrian experiments (1891–92) seem to show that reinforcing concrete with small I-beams adds greatly to the strength of the arch, but that small rods or netting are not of sufficient advantage to warrant the additional expense.* Tests made on arches of 8-ft. span gave the following results: Concrete arch, $3\frac{3}{8}$ ins. thick, $9\frac{1}{2}$ ins. rise, broke at 1130 lbs. per square foot. A Monier arch (wire netting), $1\frac{15}{16}$ ins. thick, $10\frac{1}{4}$ ins. rise, or about half the thickness of the concrete arch, failed at 1217 lbs. per square foot. Brick arch, $5\frac{1}{2}$ ins. thick, 9.85 ins. rise, failed at 885 lbs. per square foot. A hollow brick arch, $3\frac{15}{16}$ ins. thick, $5\frac{15}{16}$ ins. rise, failed at 401 lbs. per square foot. A 13-ft. span, concrete arch, $3\frac{15}{16}$ ins. thick, $15\frac{7}{8}$ ins. rise, failed at 812 lbs. per square foot. Melan arch, $3\frac{1}{8}$ ins. thick, 11.4 ins. rise, broke at 3360 lbs. per square foot. The Melan arch had I-beams $3\frac{1}{8}$ ins. deep, spaced 40 ins. apart. The structure was one year old when tested.

While there are several patented arched-floor systems, a plain concrete arch can be built by any one, and if reinforcing is desired for wide spans, plain rods or bars, small tees or channels, and various forms of netting may be used without infringing on any patents. The principal advantages of the patented arch systems lie in the matter of economy in putting the arches in place.

The concrete arch, considered as a monolithic construction, if built of stone concrete, is superior to the brick arch.

The cinder-concrete arch is inferior only in point of strength. Such an arch should be at least 4 inches deep at the crown, and the rise should be not less than one-eighth of the span. The strength of such an arch for ordinary cinder concrete would be about the same as that of a 6-inch segmental tile arch of same span, as given in table on p. 755.

All arched systems require tie-rods between the beams to take up the thrust of the arches, the same as for tile arches, see p. 804.

The Roebling Arch-floor System.—This system is now so widely known as to require but brief description. It has been

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* See *Architecture and Building*, Jan. 4, 1896.

used in many of the best buildings in the Eastern States, and has proven one of the strongest floor systems in use, and when the bottoms of the steel beams are protected as in Types 2

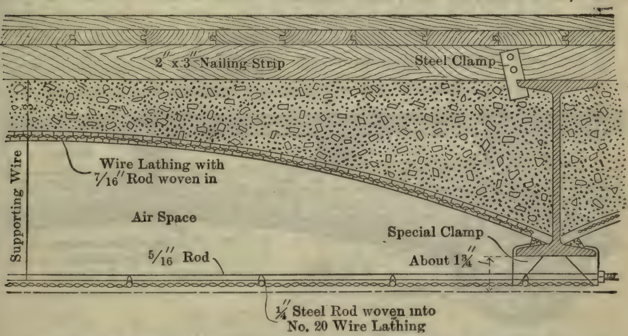


Fig. 34.—Type 1. For buildings requiring level ceilings.

and 4 is unquestionably first-class fire-proof construction. The three principle types of floor construction are shown by Figs. 34, 35, and 36. Type 3 is similar to Type 2, but has a sus-

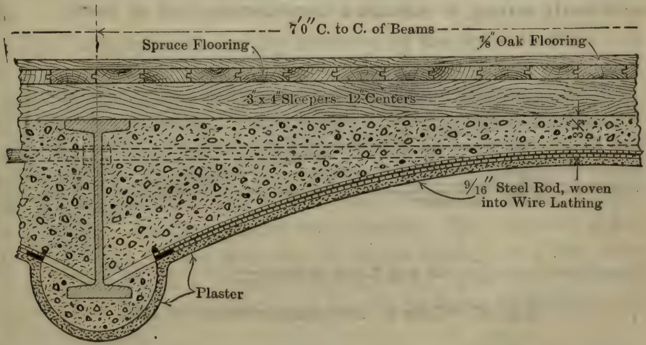


Fig. 35.—Type 2. Warehouse construction.

pended flat ceiling in addition, which may be adjusted at any level below the floor beams to admit piping, etc., as may be desired. The distinctive feature of this system is the permanent wire centering which is always erected in advance of the concreting, thus enabling the work to progress continu-

ously. The centering is made of the proper size and form at the factory, so that it is readily placed in position.

The advantages of this centering, aside from the saving over wood centres, and the rapidity with which it can be put

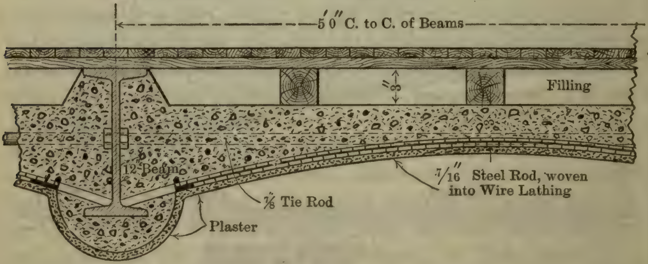


Fig. 36.—Type 2. Warehouse construction with sleepers depressed.

in place, are, that it allows the superfluous water to drip out of the concrete as soon as it is in position and it also forms a valuable safeguard against the falling of workmen, as it is sufficiently strong to sustain a considerable load of itself.

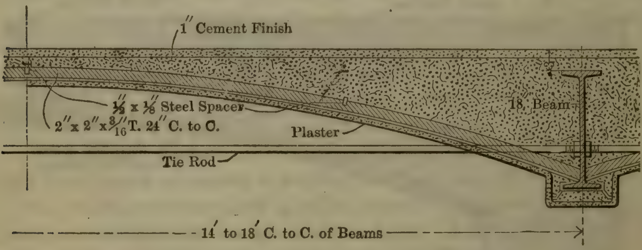


Fig. 37.—Type 4. For spans exceeding 10 feet.

Sections of the Roebling arch floor have been tested to from 1400 to 4100 lbs. per square foot without failure.

With spans of from 5 to 6 ft., the author considers that they will support 1000 lbs to the square foot with an ample factor of safety.

The maximum spans that are desirable for the different types are given in the illustrations. Type 4, Fig. 37, however,

has been installed with success up to 18 ft. between 18-inch beams.

Wide spans require a corresponding depth at the haunches, as the clear rise of the arch for Types 1, 2, and 3 should be $1\frac{1}{2}$ ins. per foot of span. In the 18-foot span, above mentioned, the clear rise above beam-flange was 16 ins. For 14-foot spans between 18-inch beams, the rise would be 14 ins.

The following table, prepared by the Roebling Construction Company, gives the weight per square foot for different spans:

When concrete is to be leveled above underside of floor beams to a height of	Maximum spacing of iron floor beams (independent of size of beams) should not exceed	Thickness of crown at centre of arch.	Weight per sq. ft. including only concrete and wire.
8″	4′ 0″	3″	33 lbs.
9″	4′ 6″	3″	34 lbs.
10″	5′ 0″	3″	36 lbs.
12″	6′ 0″	3″	41 lbs.
15″	7′ 6″	3″	47 lbs.

The weights given are for concrete to the level indicated in the first column, with a 3-inch crown and for all wire construction, including arch wire for floors and lathing for ceiling.

Flat Reinforced Floors.—The floors consist of a slab of concrete, varying in thickness according to span and load set, between the steel floor beams and reinforced near the lower surface with steel in one of the shapes referred to on p. 772, and further described under each system. For ordinary loads the thickness of the slab should be at least $\frac{5}{8}$ inch for each foot of span, with a minimum thickness of $3\frac{1}{2}$ inches. Thinner slabs have been used but they should be carefully studied for each condition.

The floor slabs are not usually of the same depth as the beams supporting them. The position of the slab will therefore determine the character of the ceiling. When the bottom of the slab is placed at or below the lower flanges a flat ceiling is obtained, and the space over the slab must be filled with some incombustible material to the underside of the flooring, thus often increasing the weight. When the slab is set at the

top flange, a paneled ceiling is obtained, unless a hung ceiling is provided.

The type of reinforcement used is generally the distinguishing feature of the different so-called systems, but the principles involved is the same in all.

Expanded Metal.—This material is now so well known as to require but little description. The diamond mesh shown

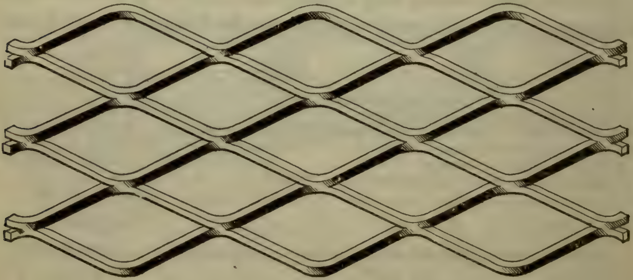


FIG. 38.—Expanded Metal, Diamond Mesh.

by Fig. 38 is used in floor construction. For this purpose the 3-inch mesh is used, the mesh being designated by the width of the diamonds. It comes in sheets 8 or 12 feet long, and varying from 3 to 6 feet in width, according to the width of the mesh. It is made from a soft, tough steel of fine texture, varying in thickness from No. 16 to No. 4 Stubbs gauge.

When used between I-beams without other reinforcement, the spans usually vary from 5 to 6 feet, although panels 11 feet

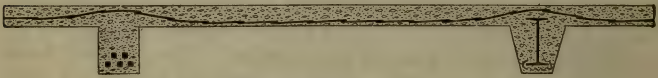


FIG. 39.—Expanded Metal Floor Construction.

wide between beams have been constructed. Of the numerous styles of floor construction possible with expanded metal reinforcement, that shown in Fig. 39 is most generally used and is recommended. At the right hand of the figure is shown the construction when steel beams are used, and at the left hand when reinforced concrete beams are employed.

The advantages of expanded metal as reinforcement are claimed to be a better arrangement in the concrete than an equal amount of material in any other form; great efficiency

in the carrying of concentrated loads, due to the obliquity of the strands; a uniform distribution of small sections at frequent intervals as preferable to larger sections at greater intervals; an increased ultimate strength and high elastic limit due to the method of manufacture, thus combining the advantages of a low-carbon steel with high ultimate strength; and a mechanical bond with the surrounding concrete.

Welded Metal Fabric.—The Clinton Wire Cloth Company manufacture a welded fabric or mesh which has been extensively used in the United States as a reinforcement for concrete construction of all kinds.

Fig. 40 shows the general style of the fabric, although the meshes and wires can be varied to an almost endless extent,

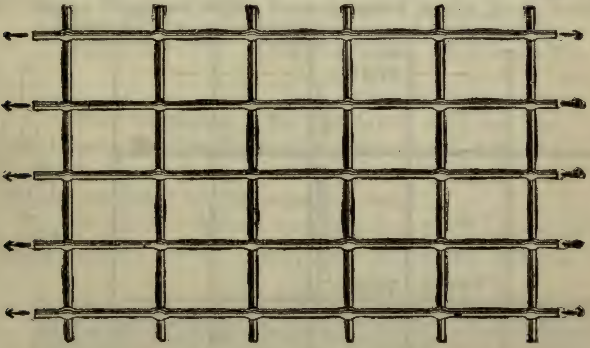


Fig. 40.

the only limit being as to mesh, 1 in. being about the closest that can be made.

The advantage claimed for this fabric as reinforcement for slab construction is that the carrying wires may be varied, both in size and spacing, to give the necessary area for any given weight and span, and the distributing or cross wires may also be varied in the same way. The direction of the wires coincides with the line of stress, so that there is no tendency to distort the rectangle of the mesh.

The cross wires, being welded to the carrying wires, are rigidly held in place and prevent the latter from slipping in the concrete.

In the meshes most commonly used the carrying wires vary from No. 10 to No. 3,* and from 1 to 4 ins. on centres, while

* Washburn & Moen gauge.

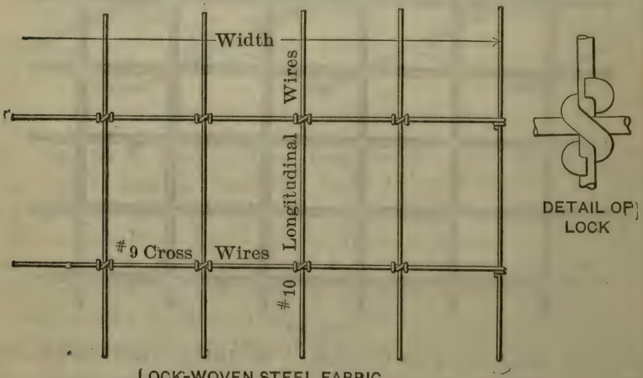
the distributing wires vary from No. 11 to No. 6, and from 3 to 12 ins. on centres.

Welded metal is manufactured in long rolls, and by its use, all joints and laps are avoided, and a floor can be made with a continuous metallic bond from wall to wall (i.e., when the mesh is laid over the top of the steel beams).

The width of rolls varies from 48 to 86 inches.

This material can be used in all the ways in which other meshes are used, and the author understands can be purchased by any responsible party. Mr. E. Lee Heidenreich states that he has obtained remarkable results with this material as to strength and lightness.

Lock-woven Fabric.*—This fabric is woven from high carbon steel wires, crossing each other at right angles, and



LOCK-WOVEN STEEL FABRIC

FIG. 41.

locked at the intersection by means of No. 9 wire twisted around the strands, as shown in Fig. 41.

The standard fabric is 56 ins. wide, and is put up in rolls of 330 to 500 lineal feet, or of any shorter lengths desired.

The longitudinal strands are No. 10 wire, B. & S. gauge, 4 ins. on centres, and the cross strands No. 9 wire, 6 ins. on centres. The standard fabric weighs two-tenths pounds per square foot.

This fabric can be woven of any gauge wire and with any

* Manufactured and for sale by W. N. Wight & Co., New York.

larger mesh, either square or oblong, that may be required. It can also be made up to 88 ins. wide.

It is sold either bright or galvanized—the galvanized costs but $1\frac{1}{3}$ cents per square yard more than the bright.

This fabric has the same advantage as the welded and tie-lock fabrics in that it can extend from wall to wall, thus making a continuous tie.

It is used to best advantage in a construction like Fig. 42, but may be used in any way suitable to an open mesh fabric.

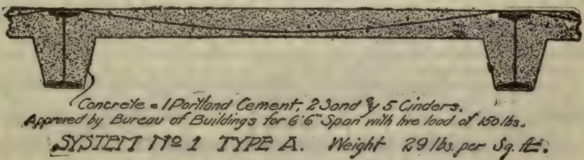


FIG. 42.

For spans exceeding 8 ft., the fabric may be used in two thicknesses, one directly over the other, and with the strands staggered, this arrangement, however, being covered by a patent controlled by the manufacturers.

Fig. 43 shows a system recommended by the manufacturers for apartment houses, hotels, etc., or for any building where

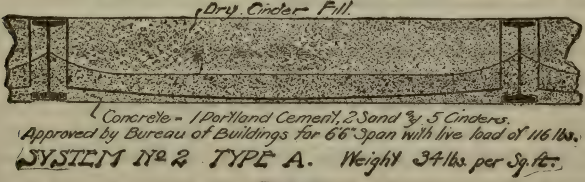


FIG. 43.

the loads are not excessive. The special advantage of this system is that it gives a level ceiling combined with strength and comparatively light weight.

The manufacturers issue a folder showing various forms of floor arches that may be constructed with their fabric. Most of these devices have been patented, but all are free to those who buy the fabric.

Strength.—Systems 1, 2, and 3 have been approved by the Bureau of Buildings of the City of New York for the spans and live loads indicated.

In Sept., 1902, six floor arches, reinforced with lock-woven fabric, were tested for strength by the Bureau of Buildings, Borough of Manhattan.* An arch like Fig. 42, 4½ ins. thick and 6-foot span, was loaded to 1452 pounds per square foot, and one of 8 ft. span, 5 ins. thick, to 1412 pounds per square foot. The latter arch had two layers of fabric.

An arch like Fig. 43, with concrete 4½ ins. thick and 6-foot span, slowly failed under 1165 pounds per square foot.

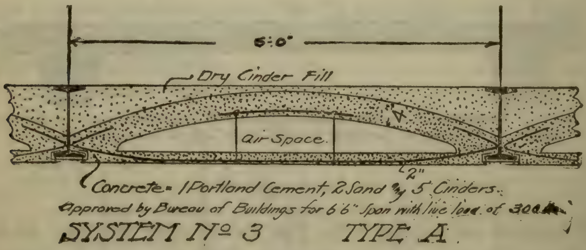


FIG. 44.

An arch like Fig. 44, 6-foot span, was loaded to 3018 pounds per square foot without breaking.

Steel Wire Reinforcement.—The American Steel and Wire Co., of Chicago, has placed on the market a wire fabric for reinforcement of fire-proof floors. Two types are being manufactured for this purpose, known as "Triangular Mesh," Fig. 45, and "Square Mesh," Fig. 46.

The triangular mesh is built up of either stranded or solid longitudinal or tension members, with the cross or bond wires running diagonally across the width of the fabric. The square mesh is similar to that made by other manufacturers of this style, but the longitudinal or tension members may be either single or stranded wires. The cross or bonded wires in this case run at right angles with the tension members.

The manufacturers recommend the use of the triangular mesh, as it is claimed that it not only affords the most even distribution of the steel, reinforcing in every possible direction, but also provides a better mechanical bond between the steel and concrete. Tests, made by the manufacturers, would seem to show that the triangular mesh have a decidedly higher ten-

* Complete details of these tests are published in *Insurance Engineering* for Oct., 1902.

sional value than the square mesh of the same weight per square foot. The cost of the two types, of equal weight, is said to be

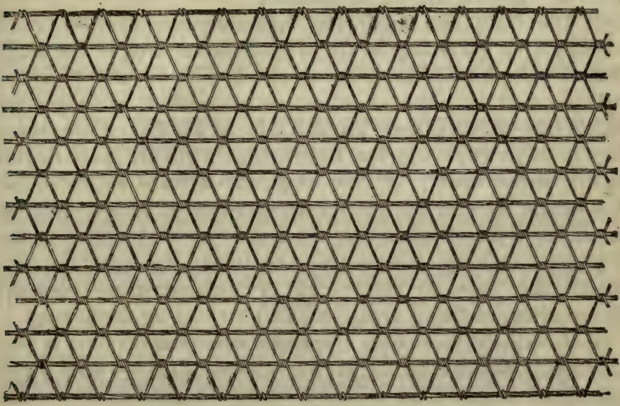


FIG. 45.

the same. The use of the stranded wire fabric is also recommended over the single wire fabric, because of the mechanical

FIG. 46.

bond afforded by the stranding in addition to the adhesion of the concrete to the steel.

For floor reinforcement this fabric is used in the same manner as any of the other fabrics previously described, and as indicated in Figs. 31, 34, 39, 42, and 43.

Of the triangular mesh the manufacturers offer 88 stock styles, giving variations in the cross-sectional area of the longitudinal wires from about 0.023 square inch to about 0.36 square inch per foot in width of fabric, or a variation in weight per square foot of fabric from about 0.2 lb. to 1.5 lbs. These styles vary in three ways: (1) The longitudinal wires are spaced either 4 or 2 inches apart; (2) the longitudinal wires consist either of a single wire or of two or three wires stranded; and (3) the cross or bond wires are either No. 14 or No. 12½ gauge. This great variety affords an opportunity for selecting very closely the cross-sectional area of reinforcement required. Special sizes, of larger areas, can be provided on application.

Of the square mesh only the single wire strand fabric is kept in stock, there being 28 styles, representing 2- and 4-inch spacing of longitudinal wires, of cross-sectional areas varying from 0.026 square inch to 0.295 square inch, and a 6- and 12-inch spacing of No. 12 cross wires. The weight of this fabric varies from 0.13 lb. to 1.15 lbs. per square foot.

Patented Systems of Flat Floor Construction. — The following systems of floor construction, while based upon the same general principles as those already described, are patented and can be used only by the patentees:

Roebling Flat Construction. —This system was introduced by the Roebling Construction Company to meet the demand for a light economical floor, with greater spans between the I-beams than is practicable for their arched system.

This flat construction is a reinforced concrete system, differing from other flat systems only in the reinforcing frame. The details of construction are quite clearly shown in Fig. 47. The main tension members consist of flat bars, usually 2 ins. in width and varying from ⅛ to ¼ in. in thickness, according to the spacing of the beams and the load to be supported. These bars stand on edge in the concrete, and are twisted at the end to lie flat on the I-beams, and are also bent around the flange. The bars are held in position laterally by means of spacers, formed from half-oval iron, with a hook at each end to fit over the bars.

It was the original intention of the patentees to apply the Roebling stiffened wire lath to the underside of the tension bars by means of lacing wire, to serve as a centering for the

concrete, and under certain conditions the wire centering is still used. When the building is to be erected in a large city, it has been found more advantageous to use wood centering, for the reason that the concrete is not as easily damaged by workmen walking over it before it has thoroughly set as when wire centering is used.

The latter, however, has important advantages when the work is to be erected in cold weather, as it permits the moisture to drip away rapidly and prevents the concrete from being

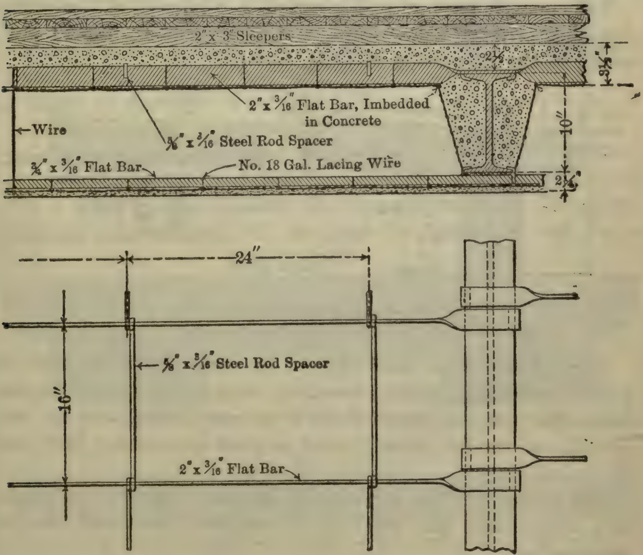


FIG. 47.—Roebling Flat Construction.

injured by freezing. In isolated places, where the lumber for centering would have to be purchased new and then disposed of at a sacrifice, the wire centering is also the more economical.

Besides the type of construction shown by Fig. 47 three other types, differing principally in the manner of supporting the steel bars, are employed.

In Type 4 the tension bars are supported on the bottom flange of the I-beams, so as to give a level ceiling between the beams. This type, however, is not desirable when the I-beams are more than 7 ins. deep.

When the distance between the steel beams is greater than 9 or 10 ft., the tension bars are bent downward so as to give a sag of about 2 ins. or more at the center of the span, as in Fig. 48, the spacers being used as in Fig. 47. Type 5 has been successfully used in spans up to 22 ft. Under ordinary conditions, however, considering both the steel work and the fire-proofing, the most economical results will be obtained when the girders are spaced from 14 to 16 ft. apart. With this system a suspended ceiling is not necessary or desirable.

The concrete used with this system is composed of high-grade Portland cement, sharp sand, and clean cinders, mixed ordinarily in the proportion of 1, $2\frac{1}{2}$, and 6.

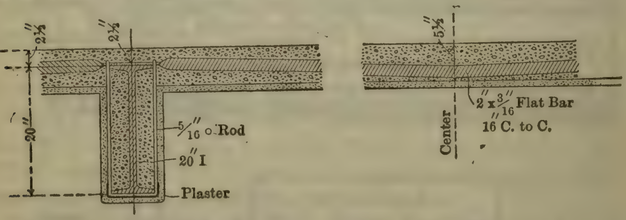


FIG. 48.—Long-span System. (Type 5.)

Adaptation.—This floor system is particularly adapted to public buildings, offices, theatres, schools, hospitals, hotels, residences, etc., or where there is no great weight to be supported, and the fire hazard is not as great as in stores, factories, etc.

The system can be successfully adapted, however, to stores and warehouses, but will require shorter spans and heavier construction.

Strength.—The Roebling Construction Company claims that Type 1 has a safe carrying capacity, with factor of safety of 4, of 200 lbs. per square foot with a span of 8 ft., and that Type 5, with a span of 16 ft., will safely support a load of 100 lbs. per square foot.

A section of floor 4 ft. 5 ins. wide and 16 ft. span, carried a total load of 17,250 lbs. with a deflection of only $\frac{7}{16}$ inch.

The Columbian System.*—This is a flat concrete system, with ribbed steel bar tension members differing from the system previously described, principally in the shape of the reinforcing

* Controlled by the Columbian Fireproofing Co., Pittsburgh, Pa.

bars, which are entirely different in shape from those used in any other system, and very much deeper. The general shape of the 1, 2, $2\frac{1}{2}$, and $3\frac{1}{2}$ inch bars is shown by the hole in the stirrup, Fig. 49; the larger sizes have more ribs, as shown by the reduced section at *A*.

The Columbian floors are made in two styles, "long span" and "short span."

The short span consists of the use of ribbed steel bars suspended from the steel beams, and supported on edge by means of steel stirrups which have the profile of the bar cut in them, as shown by Fig. 49, these bars being surrounded by and completely imbedded in concrete. This type of paneled construction is plainly shown by the sectional drawing, Fig. 50.

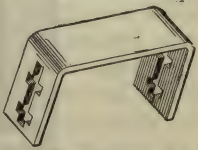


Fig. 49.—Stirrups for Bars.

A, Section of 5-inch bar (reduced).

If a level ceiling beneath the beam is required, it is constructed independently of the floor by means of solid con-

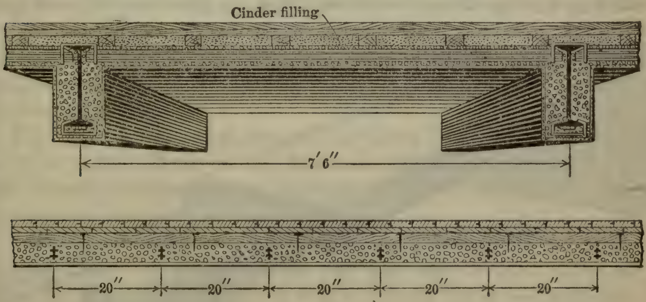


Fig. 50.

crete ceiling on lower flange of beams, wire lath expanded metal, or any other form of metallic lath upon which the plastering is applied.

In this way all portions of the steel are completely imbedded in solid concrete. The bottom flange of the beam, which is the most vulnerable point, being protected by a concrete slab, Fig. 51, with insulating air space. Three sizes of bars are used for this floor construction, viz., $2\frac{1}{2}$, 2, and 1 ins., the maximum spacing of the bars being 24 ins. The carrying capacity of this floor is given by the table on page 790.

The most economical spacing of floor beams for this type will usually be 6 ft. for hotels, apartment-houses, and office-buildings, using 1-inch bars, and from 6 to 9 ft. for greater floor loads,

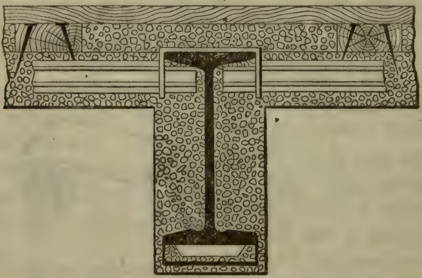


FIG. 51.—Showing Protection of Bottom Flange.

using 2- and 2½-inch bars, depending upon the load required to be carried.

This floor may be finished on top in the usual way by imbedding nailing strips in cinder filling, or the floor strips may

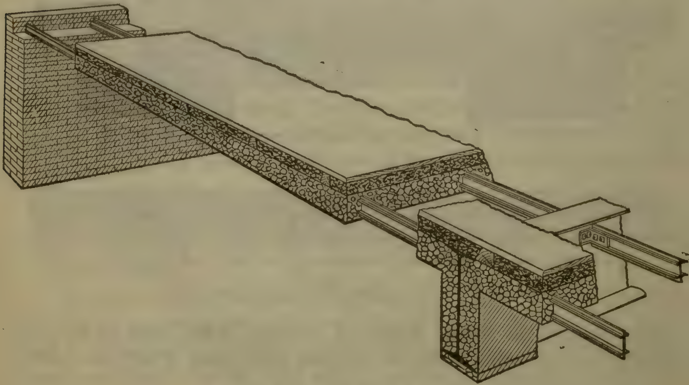


FIG. 52.—Long-span System.

be nailed directly to the concrete floor, and the filling omitted. The economy of this type of construction is that wall channels are not required, and beams may be spaced up to 9 ft. centre to centre.

Long-span Construction.—In the second type of this construction, or what is commonly known as "long span," the

rolled and ribbed steel bars are imbedded in the concrete, as in the short span, and either hung in specially formed stirrups or framed directly to the beam, as shown by Fig. 52, these bars being anchored at intervals into the wall, and forming a continuous tie across the entire floor of the building, thus making of the entire floor a monolith of reinforced concrete. This permits of the elimination of the floor beams between girders, the monolithic slab of concrete and steel taking their place. In this way a level ceiling is obtained between girders, so that increased head room can be obtained with the same amount of masonry, or the same head room with decreased height of masonry. This is possible because of the fact that the extreme thickness of this floor construction on a span of 20 ft. between beams, is but $6\frac{1}{2}$ ins., whereas for a span of 15 ft., with lighter load, the thickness of concrete may be reduced to 5 ins. The sizes of bars used in this type of construction are $3\frac{1}{2}$, $4\frac{1}{2}$, 5, and 6 ins., giving respectively 5, $5\frac{3}{4}$, $6\frac{1}{2}$, and $7\frac{1}{2}$ ins. of concrete.

Carrying Capacity.—The table on page 790 gives the loads that the Columbian Company guarantees their various forms of floors will carry safely. They call attention to the line of deflection of their 1, 2, and $2\frac{1}{2}$ ins. bars, below which the loads given for the respective spans should only be used for ceilings or pitched roofs.

This table is compiled from actual tests on sections of floor, using a safety factor of 4. Bars can be spaced down to 18 ins., thereby increasing the strength of floor, but 2 ft. is the maximum spacing.

This construction is especially strong in resisting drop or jarring loads. A ram weighing 238 lbs. was dropped from a height of 8 ft. on the centre of a span several times and without perceptible effect on the floor. In case of overloading the floor will not fail suddenly, but the construction will gradually bend, thus giving warning of danger. It also offers a great resistance to concentrated loads, which occur at times in buildings, to an extent beyond what the floor is designed to carry.

After a fire- and water-test of three hours' duration made on this system in Boston, published in the *Engineering News* of Nov. 21, 1901, this floor on a span of 11 ft. 3 ins. carried 1650 lbs. with deflection of only $1\frac{3}{8}$ inch. Before the floor slab began to show any sign of failure, loading had to be stopped

SAFE LOADS FOR COLUMBIAN FLOORS.

(In addition to the weight of floor construction.)

Distance between Supports.	Floor Loads in Pounds, Uniformly Distributed. Bars 24" on Centres.				Distance between Supports,	Floor Loads in Pounds, Uniformly Distributed. Bars 24" on Centres.		
	6" Bar.	5" Bar.	$4\frac{1}{4}$" Bar.	$3\frac{1}{2}$" Bar.		$2\frac{1}{2}$" Bar.	2" Bar.	1" Bar.
Feet.					Feet.			
12	412	362	312	275	5	400	340	290
13	346	306	265	235	6	275	242	200
14	296	260	230	200	7	200	178	140
15	256	226	200	175	8	150	135	112
16	225	200	175		9	125	100	70
17	200	175	125		10	100	70	
18	177	150			11	80		
19	140	100						
20	100	80						
Concrete thickness	$7\frac{1}{2}$"	$6\frac{1}{2}$"	$5\frac{3}{4}$"	5"		4"	$3\frac{1}{2}$"	3"

on account of the fact that the walls of the test hut which carried the floor began to crack.

Dovetail Corrugated Sheets (Ferroinclave).—Sheets of thin steel corrugated so as to form dovetail grooves have

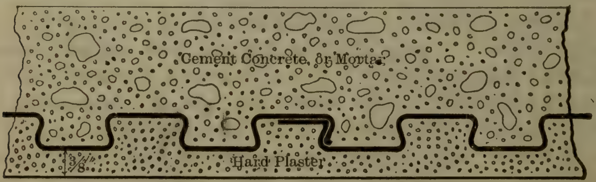


FIG. 53.—Ferroinclave.

been used by several parties, as a reinforcement and centering for concrete steel, the dovetailing serving to unite the sheets to the concrete.

The Brown Hoisting Machinery Company of Cleveland have patented, under the name of ferroinclave, a tapered corruga-

tion which is small enough to hold hard mortar, and hence can be plastered on the under side, which is a great advantage. Fig. 53 shows a partial section of the ferroinclave corrugated sheets, reduced a little more than one half, the actual size of the corrugations being $\frac{1}{2}$ in. in depth, and 2 ins. centre to centre of corrugation, with an opening between the edges of $\frac{7}{8}$ inch.

The tapering of the corrugations is also an advantage, especially for roofs, as it enables the sheets to be lapped at the end joints, so as to make a roof that will be absolutely tight, even if water should penetrate the cement coating.

The principal advantages of corrugated sheets for floor construction lies in their ability to sustain the concrete (with moderate spans) before it has set, thus saving the cost of centering and the time required in putting it in place. This advantage, however, appears to be offset by the high cost of the sheets when they have to be shipped, which brings the cost of the completed floor fully as high as the average of the reinforced concrete systems, and higher than some.

For roofs, however, it makes a light and cheap construction, as the total thickness need not exceed $1\frac{1}{4}$ ins. for spans of 4 ft. 10 ins., and it only requires a coat of asphaltic paint over the cement to make the roof watertight.

With a good coat of hard plaster or gauged mortar on the under side, the iron will not be affected by heat until a considerable time has elapsed, and even if the mortar on the under side should be more or less dislodged by the streams of water, it can be replaced, at a very slight expense. Another advantage of ferroinclave for roofs is that the building can be covered and made watertight in the most severe winter weather and the cement can be applied during the following spring.

Ferroinclave is made in sheets 20 ins. wide and up to 10 ft. in length and usually of No. 24 gauge.

For roofs the ferroinclave is attached to purlins in the same manner as iron roofing, the most economical spacing of the purlins being 4 ft. $10\frac{1}{2}$ ins. centre to centre, which accommodates sheets 10 feet long with an end lap of 3 inches.

For the cement top coat, a mixture of one part Portland cement to two parts sand, applied to a thickness of $\frac{3}{8}$-inch above the top of the sheets, is sufficient for roofs. For floors a rich gravel or crushed stone concrete should be used, the thickness being governed by the span and loads to be supported.

The following table shows the ultimate strength of No. 24

ferroinclave with different thicknesses of concrete, as determined by actual tests with sheets 20 ins. wide and 4 ft. 10½ ins. span:

Thickness of 1 to 2 mortar above the metal..	1½″	2″	2½″	3″	3½″	4″
Ultimate strength in lbs. per square foot (span 4 feet 10½ inches)	615	915	1220	1560	1860	2120

A factor of safety of 6 should be ample for ordinary loads.

About a million square feet of ferroinclave has thus far been used for floors, roofing, and side walls. It is especially adapted for the walls, roof, and floors of large manufacturing plants, and may be used to advantage for partitions, gutters, stair treads, vats, water-closets partition, and fire-proof doors.

Berger's "Multiplex Steel Plate."—Fig. 54 shows a section of a corrugated steel plate manufactured by the Berger

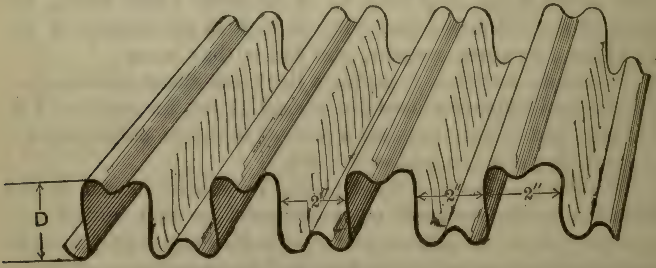


FIG. 54.

Manufacturing Company for floor and roof construction, the plate being an invention of G. Fugman, architect. As shown by the illustration, it consists of a series of vertical corrugations of sheet steel, painted or galvanized, ending at the top and bottom in three half circle arches, separating the vertical sides of the corrugations from each other, and giving stiffness to the top and bottom of the plate.

This plate is made with depths, *D*, of 2, 2½, 3, and 4 ins., and a uniform width of 2 ins. between manifolds. The sheets are made at present in lengths up to 8 ft., but the company expects to make greater lengths in the near future.

It can be made of any gauge of steel from No. 16 to No. 24, but No. 18 is as heavy as would generally be required.

For floors and roofs, the corrugated plate is laid on top of the beams and the top portion filled with concrete and leveled

off about 1 inch above the plate. For wood floors the nailing strips may be imbedded in the concrete, the bottom of the strips being raised only about ½ inch above the top of the plate.

This construction is very light and strong and requires no centering. It cannot be plastered underneath, however, and where a plaster ceiling is required it must be constructed independently of the plate by means of furring strips and metal lath.

The weight of the 4-inch plates, No. 18 gauge, with rock concrete leveled 1 in. above top of plate, is about 39 lbs. per square foot, and the safe load for an 8-foot span is given at 430 lbs. per square foot.

While this floor has several practical advantages, it cannot be considered as thoroughly fire-proof, on account of the metal being exposed on the bottom. However, with a plastered ceiling underneath, the iron would probably not be affected by any ordinary fire before the latter could be controlled.

Sectional Systems.—Several types of fire-proof floor construction have been devised that contemplate the use of factory-made units to be taken to the building and set in place between the steel beams without centering, or at least very little centering. The advantages of such system, if practicable, are obvious. Few of these constructions have, however, had commercial success. The systems of Hyatt, Pelton, and De Mann have disappeared from the market.

Thacher Floor Unit.—The so-called Thacher Floor Unit, shown in Fig. 55, is the product of the Concrete-Steel Engi-

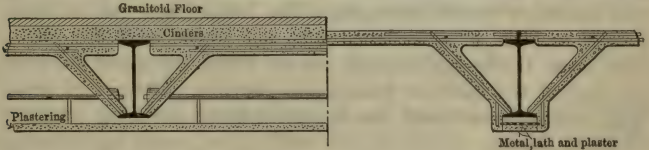


FIG. 55.

neering Co., of New York. These units are manufactured in suitable moulds of any convenient width for handling (usually about 2 feet) and allowed to set, preferably about a month or more, until they gain nearly full strength. Then they are set in place and dovetailed together with cement mortar. This design gives a very light, strong and economical floor and may be made for any desired load or span.

Waite's Concrete Beam.—In Fig. 56 is shown a type of sectional floor construction that has been used in a number of buildings by the Standard Concrete-Steel Co. of New York. The floor construction consists of a series of concrete I-beams 10 or 12 inches in depth, supported on the lower flanges of the steel beams, which are spaced 5 to 7 feet apart. The concrete beams are set about 12 inches apart and the space between the lower flanges is filled in with a cinder concrete of same composition as the I-beams. This produces a level ceiling. On the tops of the concrete beams is placed a metal fabric of small mesh on which a lean concrete slab is laid. This makes a comparatively light floor construction, because of the large spaces between concrete beams.

The concrete I-beams are cast at the shop and allowed to harden before being sent to the job. In the lower flange is

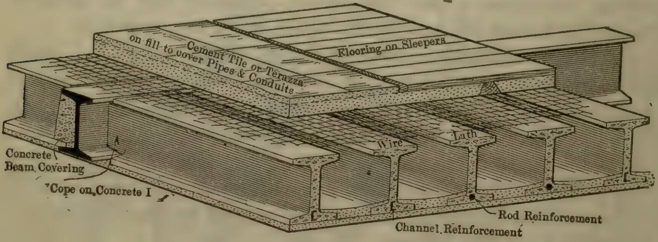


Fig. 56.—Concrete I Beams.

inserted, as shown, a steel rod or other small shape to furnish the necessary tensile strength. The beams are cast to their proper lengths, in accordance with the plans; and any slight variations at the job is made up by filling the space between the end of the concrete beams and the web of the steel beam with the concrete used for the steel beam covering between concrete beams.

A similar construction, consisting of a series of T-beams, with lower flanges $1\frac{1}{2}''$ thick and $12''$ wide, and stems $2''$ thick and $12''$ deep, of 1:4 cinder concrete, reinforced with $\frac{3}{16}''$ rods near the flanges, without floor finish of any kind, successfully withstood the fire, water, and load test of the New York Bureau of Buildings (see p. 745) after having been constructed 28 days.

Disadvantages of Sectional System.—The reason that the sectional systems have not found favor is the necessity

for a fairly uniform spacing of beams throughout a structure, which is generally impracticable. Furthermore, the casting of the parts is generally not commercially successful, as the forms, although they may be used repeatedly, are more expensive than the centering on the job, and the concrete must generally be a richer and more carefully prepared mixture, to stand handling. Even with all possible care, the breakages in transportation will be considerable.

Merrick System.—A floor construction deserving of further notice as fire-proof is the Merrick System, described in Chap. XXIV, and shown in Fig. 24, on p. 881*v*. The construction there described consists of stone concrete with a cinder concrete ceiling plate. The same construction, entirely of 1: 2: 5 cinder concrete between steel beams 7′ 6″ apart, has been approved by the New York Bureau of Buildings for loads of 150 lbs. per square foot after the required fire, water, and load test. (See p. 745.) The concrete beams were 3 ins. thick, reinforced at the bottom with a $\frac{1}{4}$ in. rod; the ceiling plate was 2 ins. thick, and the floor plate $1\frac{1}{2}$ ins. thick; the total depth from ceiling surface to top of floor plate was $13\frac{1}{2}$ ins.; and the concrete beams were spaced 24 ins. on centres.

Protection of Girders and Beams.—No form of floor construction can be considered thoroughly fire-proof unless it includes a protection of the lower flanges of all steel beams and girders, or provides for the protection of all steel used in its construction or support. The material used for the protective covering is generally the same as that used in the floor construction itself. The principal materials are terra-cotta, either dense, porous, or semi-porous, and concrete, either of cinders, stone, or slag. Plaster compositions have also been used, but are not recommended. (See p. 743.)

Beam protection, where the floor construction incases the sides of the beam, as in Figs. 16, 19, or 43, should never be less than 1 inch thick. Where paneled ceilings are used, that is, where the lower portion of the beams is below the lower side of the floor construction, as in Figs. 17, 42, or 51, the protection should be increased to at least $1\frac{1}{2}$ inches at all points.

Terra-cotta Protection.—In the case of terra-cotta, two types of protection are in use. In one case the blocks incasing the lower flanges of the steel beam meet at the center of the flange; in the other, they simply turn under the edges of the flanges and hold a flat tile with beveled edges against the lower side

of that flange. The former is the better method, as in the latter the danger of breakage of the part extending under the flange, is supplemented by the possibility of omitting the flat protection tile. The blocks incasing the lower flange may be the skew-backs of the arch, or they may be separate blocks.

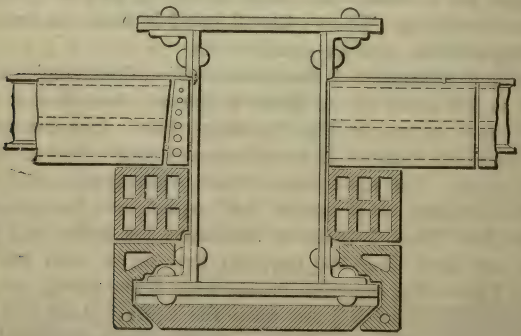


FIG. 57.

Different forms and conditions are illustrated in Figs. 14, 25 27, 19, 26, 17, 20, 21, 22, 23, and 16. Fig. 31 shows the entire depth of beams protected by blocks on either side.

Girders, where they project below the ceiling line, as is commonly the case, are much more exposed to the effects of fire

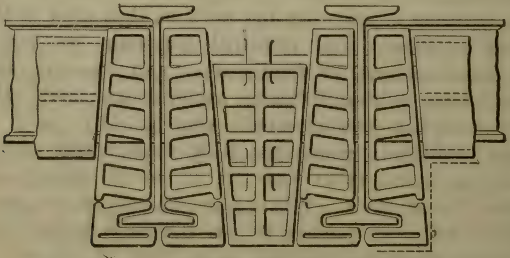


FIG. 58

and water than the floor beams, and should, therefore, have the most efficient protection. As a rule, such girders should be provided with not less than 4 inches of terra-cotta protection at the sides and $1\frac{1}{2}$ inches of solid tile under the bottom with a space of $\frac{1}{4}$ inch between the tile and the beam.

Figs. 57 and 58 are typical of the latest methods of protecting girders by means of hollow tile. The bottoms of the skews (Fig. 57) are prevented from spreading by wire ties placed in the end joints between the soffit tile and hooked into the round holes in the skew-backs. Single-beam girders are usually protected, as shown by Figs. 26 and 59, the latter figure showing

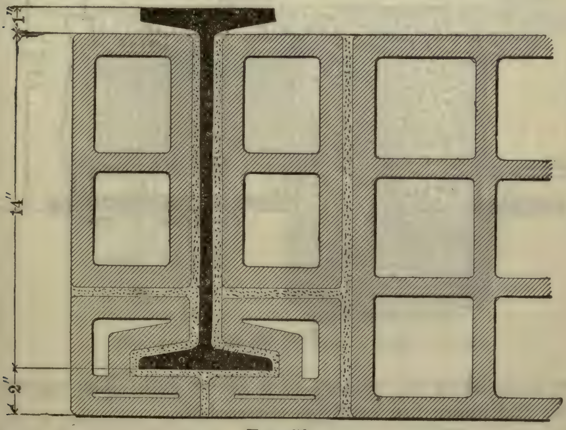


Fig. 59.

more particularly the protection of a beam at the side of an opening in the floor.

Concrete Protection.—A more thorough incasing of the webs and lower flanges of beams and girders can be accomplished by the use of concrete. The superior fire-proof character of cinder concrete makes it the best material for that purpose. If of sufficient thickness and properly applied, it will hold securely around the flanges of beams and girders without reinforcement. But where it is less than 2 inches thick wire or metal lath wrapped around the flange should be imbedded in it. Different forms of concrete protection are shown in Figs. 35, 36, and 37.

In Fig. 51 the soffit of the beam is protected by a concrete slab with an insulating air space. This method is one that may be advantageously used for the protection of girders. A fire test made in the Butterick Building, New York, on this form of girder protection thoroughly established its efficiency.

A typical girder protection in cinder concrete is shown in the Roebling method in Fig. 60.

Hung ceilings are sometimes used as the protection for the steel beams. This is very bad practice, as these ceilings are

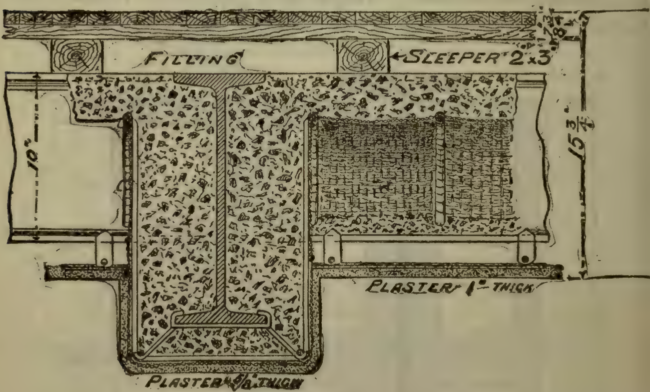


FIG. 60.

more than likely to collapse in a severe fire. The experience in the Baltimore fire confirms this belief.

Trusses.—Where steel trusses are used to support the roof or several stories of a building it is very important that they be protected, not only from heat sufficient to warp them, but so that they will not expand sufficient to affect the vertical position of the columns by which they are supported.

The following description of the covering of the trusses in the new Tremont Temple, Boston, furnishes a good illustration of the way in which this should be accomplished:

"The steel girders were first placed in terra-cotta blocks on all sides and below, these blocks being then strapped with iron all around the girders, and upon this was stretched expanded metal lathing, covered with a heavy coating of Windsor cement; over this comes iron furring, which receives a second layer of expanded metal lath, the latter, in turn, receiving the finished plaster. There is, consequently, in this arrangement for fire protection, first, a dead air-space, then a layer of terra-cotta, a Windsor cement covering, another dead-air space, and finally, the external Windsor cement."

Numerous shapes of terra-cotta tiles are made for casing the

structural shapes commonly used in steel trusses. Some of these are shown by Fig. 61. The tiles should always be secured in place by metal clamps passing entirely around the envelope, or better still, by wrapping with wire lath. The tiling should then be plastered with hard wall plaster.

Trusses may also be fire-proofed by completely incasing the several members in cinder concrete, either with or without metal reinforcement.

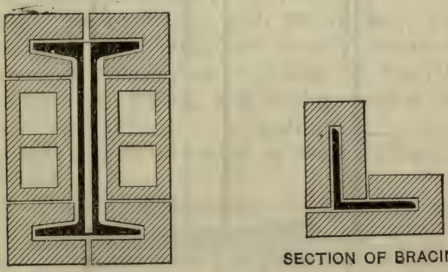


FIG. 61.—Tiles for Protecting Steel Trusses.

Where trusses are to be fire-proofed, the additional weight must be provided for in the strength of the truss.

Steel Framing for Fire-proof Floors.—Before the framing plans of a building can be made, it is necessary to decide, in a general way, upon the system of floor construction or fire-proofing that will be employed; thus if any of the long-span systems, such as the Herculean, Johnson, and many of the concrete systems, is to be adopted, the girders should be spaced so that the floor construction will span between them, without floor beams, as shown in Fig. 62, while if an ordinary flat tile arch is to be used, floor beams will be required, spaced from $5\frac{1}{2}$ to 9 ft. apart, and these beams must be supported by girders, as indicated in Fig. 63.

When there are no floor beams, a strut beam should be riveted between the columns, as in Fig. 62, to hold the latter in place during erection and to stiffen the building.

It should be remembered that with floor beams spaced not over 7 ft. from centres, almost any system of floor construction may be employed, while if the floor beams are omitted, one must select from but a few systems.

With any form of filling between beams or girders, less steel

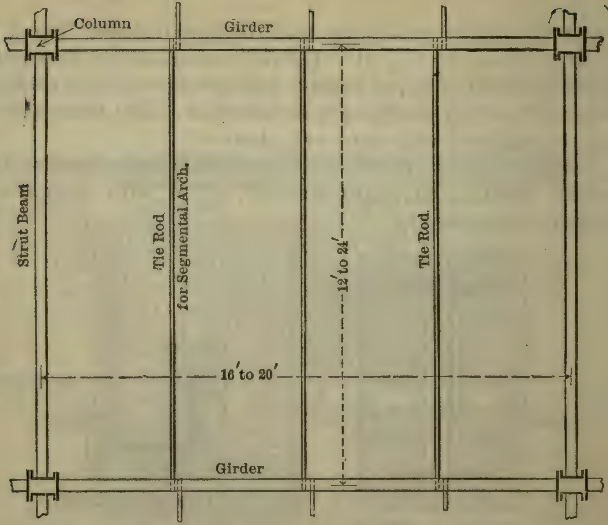


FIG. 62.—Typical Framing for Long-span Constructions.

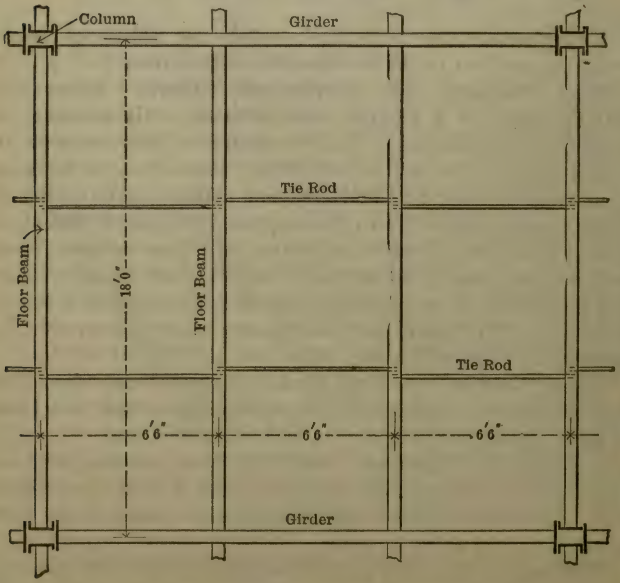


FIG. 63.—Typical Steel Framing for Short-span Arches.

will be required for moderate spans of beams or girders than when they are excessive.

Computations for the Steel Framing.—The computations for the steel beams and girders of a fire-proof floor are very much the same as for a wooden floor, viz., first estimating the load or loads which any given beam will be required to support and then finding the necessary size of beam to support the load. The dead load for any fire-proof floor may be estimated with sufficient accuracy by means of the data given in this chapter in connection with the different systems of floor construction. The dead load should include weight of beams, weight of fire-proofing, including all concrete filling, weight of plastering, furring, and lathing, nailing strips and flooring.

The live loads may be estimated by means of the data given in Chapter XXI.

Example.—The best arrangement for the posts in a retail store is 18 ft. on centres in one direction, and 19 ft. 6 ins. in the other. It is decided to run the girders as shown by Fig. 63, and to put a beam opposite each column and two between; what size of beams and girders will be required, using an ordinary end-arch construction between the beams?

Ans.—From the table on page 762, we find that the least depth of arch which it is desirable to use, is 10 in., but as we will probably have to use 12-inch beams it will be better to figure on a 12-inch arch, as this will give less filling on top. The weight of the 12-inch arch will be about 39 lbs. per square foot. We shall probably require 2 ins. of concrete filling on top, which will weigh 16 lbs., and $1\frac{1}{2}$ ins. of light filling between nailing strips, weighing, say 9 lbs. The flooring and nailing strips will weigh about 4 lbs., the plastering on ceiling 5 lbs., and we must allow at least 6 lbs. per square foot for the weight of the beams themselves. These make a total dead weight of 79 lbs. per square foot. The live load for a retail store should be taken at 150 lbs. per square foot—making a total load per square foot on the beams of 229 lbs. The total load that each beam must be capable of supporting will be $6\frac{1}{2}' \times 18' \times 229$ lbs. $= 13.4$ tons, which is assumed to be uniformly distributed. From the table, p. 516, we find that this load, with a span of 18 ft., will require either a 12-inch, 50-lb. beam, or a 15-inch, 42-lb. beam. The latter will be both stronger and cheaper, but will increase the thickness of the floor by 3 ins. and require additional filling.

The girder must support two concentrated loads of 13.4 tons each. On p. 514 it is stated that when a beam supports two equal loads applied one-third of the span from each end, the equivalent uniformly distributed load may be found by multiplying one load by $2\frac{2}{3}$. Multiplying 13.4 by $2\frac{2}{3}$ we have 35.73 tons as the equivalent distributed load on the girder, to which should be added the weight of the girder. This requires a 24-inch 80-lb. beam.

If instead of using tile arches between beams, $6\frac{1}{2}$ ft. apart, we conclude to use the Herculean or Johnson construction spanning from girder to girder, we should frame our floor as in Fig. 62. For this span we would require 10-in. tile, weighing 55 lbs. per foot. Allowing 8 lbs. for 1 in. of concrete, 9 lbs. for filling, 4 lbs. for flooring and strips, and 5 lbs. for plastering, we have 81 lbs. as the dead load per square foot (we have added nothing for weight of girder, as this will be fully offset by portions of floor not loaded). The live load per square foot will be 150 lbs. as before, and the total load to be supported by the girder $18' \times 19'\ 6'' \times 231$ lbs. $= 40.5$ tons, which will require a 24-inch 80-lb. beam; hence by this arrangement we save the weight of the floor beams, but a 6-inch strut beam should be placed between the columns, as in Fig. 62. The calculations for any other floor construction are made exactly as above, the variations being only in figuring the dead weight of the construction.

Tables for Floor Beams.—It is a difficult matter to prepare tables showing the size of steel beams required for fire-proof floors that may be generally used, for the reason that such beams are often irregularly spaced, and because of the wide variation in the dead loads. The following tables, however, may be used in making approximate estimates and in checking the computations for any particular floor. The sizes of I-beams given may be safely used where the total live and dead load does not exceed the value given in the headings. The total load should include sufficient allowance for the weight of any partitions that the floor beams may be called upon to support.

Tie-rods.—In all segmental arches and such other types in which a thrust is exerted against the beams, tie-rods must be provided to prevent the beams from being pushed apart, and especially to prevent the outer bays from spreading. They should run from beam to beam from one end of the building to the other. If the outer arches spring from an angle, as in

TABLE I.—SIZE AND WEIGHT OF I-BEAMS FOR FLOORS IN OFFICES, HOTELS, AND APARTMENT-HOUSES.

Total load, 120 pounds per square foot.

Span of Beams in Feet.	Distance between Centres of Beams.									
	4½ Feet.		5 Feet.		5½ Feet.		6 Feet.		7 Feet.	
	ins.	lbs.	ins.	lbs.	ins.	lbs.	ins.	lbs.	ins.	lbs.
10	6	12¼	6	12¼	6	12¼	6	12¼	7	15
11	6	12¼	6	12¼	7	15	7	15	7	15
12	6	12¼	7	15	7	15	7	15	8	18
13	7	15	7	15	7	15	8	18	8	18
14	7	15	8	18	8	18	8	18	9	21
15	8	18	8	18	8	18	9	21	9	21
16	8	18	9	21	9	21	9	21	10	25
17	9	21	9	21	9	21	10	25	10	25
18	9	21	9	21	10	25	10	25	12	31½
19	9	21	10	25	10	25	10	25	12	31½
20	10	25	10	25	12	31½	12	31½	12	31½
21	10	25	12	31½	12	31½	12	31½	12	31½
22	10	25	12	31½	12	31½	12	31½	15	42
23	12	31½	12	31½	12	31½	12	31½	15	42
24	12	31½	12	31½	12	31½	15	42	15	42
25	12	31½	12	31½	15	42	15	42	15	42

TABLE II.—SIZE AND WEIGHT OF I-BEAMS FOR FLOORS IN RETAIL STORES AND ASSEMBLY ROOMS.

Total load, 200 pounds per square foot.

Span of Beams in Feet.	Distance between Centres of Beams.									
	4½ Feet.		5 Feet.		5½ Feet.		6 Feet.		7 Feet.	
	ins.	lbs.	ins.	lbs.	ins.	lbs.	ins.	lbs.	ins.	lbs.
10	7	15	7	15	7	15	8	18	8	18
11	7	15	8	18	8	18	8	18	9	21
12	8	18	8	18	9	21	9	21	9	21
13	8	18	9	21	9	21	10	25	10	25
14	9	21	9	21	10	25	10	25	12	31½
15	9	21	10	25	10	25	12	31½	12	31½
16	10	25	10	25	12	31½	12	31½	12	31½
17	10	25	12	31½	12	31½	12	31½	12	40
18	12	31½	12	31½	12	31½	12	40	12	40
19	12	31½	12	31½	12	40	12	40	15	42
20	12	31½	12	40	12	40	15	42	15	42

TABLE III.—SIZE AND WEIGHT OF I-BEAMS FOR FLOORS IN WAREHOUSES.

Total load, 270 pounds per square foot.

Span of Beams in Feet.	Distance between Centres of Beams.									
	4½ Feet.		5 Feet.		5½ Feet.		6 Feet.		6½ Feet.	
	ins.	lbs.	ins.	lbs.	ins.	lbs.	ins.	lbs.	ins.	lbs.
10	8	18	8	18	8	18	9	21	9	21
11	8	18	9	21	9	21	9	21	10	25
12	9	21	9	21	10	25	10	25	10	25
13	10	25	10	25	10	25	12	31½	12	31½
14	10	25	12	31½	12	31½	12	31½	12	31½
15	12	31½	12	31½	12	31½	12	31½	12	40
16	12	31½	12	31½	12	31½	12	40	12	40
17	12	31½	12	40	12	40	12	40	15	42
18	12	40	12	40	15	42	15	42	15	42
19	12	40	15	42	15	42	15	42	15	42
20	15	42	15	42	15	42	15	45	15	55

Fig. 13, the tie-rods in this bay should be anchored into the wall with large plate washers.

The tie-rods should be located in the line of thrust of the arch, which is ordinarily below the centre of the beam, and in some cases near the bottom flange.

If their appearance is objectionable, they should be hidden by a hung ceiling.

For constructional purposes tie-rods are desirable in all types of floor construction, even though the floors do not exert a thrust on the beams.

As a rule tie-rods are proportioned and spaced by some "thumb rule" rather than by actual calculations of the thrust. For the interior arches this practice is probably safe enough, but for outside spans, and particularly for segmental arches, the thrust of the arch should be computed and the rods proportioned accordingly. The spacing of the rods is generally taken at eight times the depth of the supporting beam, but never more than 8 feet. For interior flat tile arches, the following rule can usually be safely followed: For spans of 6 ft. and under, use $\frac{3}{4}$-inch rods spaced about 5 ft. apart; for 7-ft. spans, use $\frac{7}{8}$-inch rods, 5 ft. centre to centre, and for a 9-ft. span, $\frac{7}{8}$-inch rods, 4 ft. centre to centre.

The horizontal thrust of an arch may be found by the following formula:

$$T = \frac{3wL^2}{2R},$$

in which T = pressure or thrust in pounds per lineal foot of arch;
w = load on arch in pounds per square foot, uniformly distributed;
L = span of arch in feet;
R = rise of segmental arch, or effective rise of flat arch in inches.

The rise of a segmental arch is measured from the springing line to the soffit of the arch at the centre. For flat hollow-tile arches, the effective rise may be figured from the top of the beam flange to the top of the tile. As the tiles usually project from $1\frac{1}{2}$ to 2 ins. below the bottom of the beam, the effective rise will be from 2 to $2\frac{1}{2}$ ins. less than the thickness of the arch.

For the interior arches of a floor, w may be taken for the live load only, but for the exterior arches, w should include both the full dead and live load.

Having found the thrust of the arch, the spacing of the rods (of any particular size) may readily be determined by dividing the safe load given for that size of rod in the table on p. 340 (allowing 15,000 lbs. unit stress) by the thrust. The result will be the spacing in feet.

Example.—What size of tie-rods and what spacing should be used for the floor construction described on p. 801?

Ans.—The depth of the tile arch is 12 ins., the dead load 79 lbs., and the live load assumed at 150 lbs. The span between beams is $6\frac{1}{2}$ ft. Then for the interior arches, $w=150$ lbs., $R=12-2\frac{1}{2}=9\frac{1}{2}$ ins. and $L=6\frac{1}{2}$, and $T=\frac{3\times 150\times 42.25}{2\times 9\frac{1}{2}}=1000$ lbs. The strength of a $\frac{3}{4}$-inch rod, not upset, at 15,000 lbs. is (from p. 340) 4500 lbs. Dividing by 1000 we have $4\frac{1}{2}$ ft. as the spacing.

The strength of a $\frac{7}{8}$-inch rod is given as 6300 lbs., which would admit of a spacing of 6.3 feet.

For the outer spans, w should be taken at $150+79=229$ lbs., when T will equal $\frac{3\times 229\times 42.25}{2\times 9\frac{1}{2}}=1526$ lbs.

For this thrust we should use $\frac{7}{8}$-inch rods spaced about 4 ft. 2 ins. centre to centre.

Load Tests.—It may be desirable at times to test fire-proof floors after the same have been installed. The same precautions should be observed as for tests on reinforced concrete construction, described on p. 882*p*. If it is desired to determine

from such tests the ultimate strength, a section of the floor of a width equal to the span should be cut loose from the rest and loaded to destruction, the supporting steel beams being shored up during the test. The safe working load is found by dividing the breaking load by the proper factor of safety.

Roof.—Flat roofs are constructed in the same way as the floors, except that the beams and girders are set so as to give a slight pitch to the roof, for draining the water. As the roof-loads are usually less than the floor-loads and there are no partitions to be supported, the arches or roof-panels are usually considerably lighter than the floor-panels, but the general construction is practically the same for both.

When the roof is formed of reinforced concrete, the beams should be set so that the concrete will give the desired inclination to the roof, with a nearly uniform thickness, as this reduces the amount of concrete required, and also the weight.

If the roof is to be covered with tin or copper, nailing-strips should be imbedded in the concrete, the same as for wooden floors, and the entire roof sheathed, as it is claimed that tin or copper laid over terra-cotta or concrete will rust out in a few years.*

Gravel or tile roofs require no woodwork of any kind.

Whether terra-cotta or concrete is used for the roof panels, the sides and bottoms of the steel beams and girders should be efficiently protected, and all columns or other structural metal in the roof space should also be well protected. In the ordinary building, having stair or elevator wells, the roof and upper ceiling are likely to be more severely tested by heat, in case of fire, than any of the floors below, and experience has shown that this is often the poorest protected portion of the building.

Pitched Roofs.—Pitched roofs may be constructed in various ways, according to the material that is to be used and the kind of roofing that is to be employed.

When terra-cotta is to be used for the fire-proofing, the most common method of construction is to frame the roof with I-beam rafters and T-iron purlins, set horizontally, and spaced 1 inch more than the length of the tile. Between the tees, book or roofing tiles are placed as in Fig. 64, and the roofing is applied directly to the surface of the tile. If the roofing is to be

* Freitag, page 288.

of slate or clay tiles, solid porous terra-cotta blocks should be used, between the tees, as the solid blocks hold the nails better than the hollow tile. The same construction may be used for a flat roof, but on account of the expense of the tees it will usually be more expensive than the construction above described, and not as strong or desirable. With the construction shown in Fig. 64, it is impossible to efficiently protect the bottom of the tees from the effects of heat by any economical method.

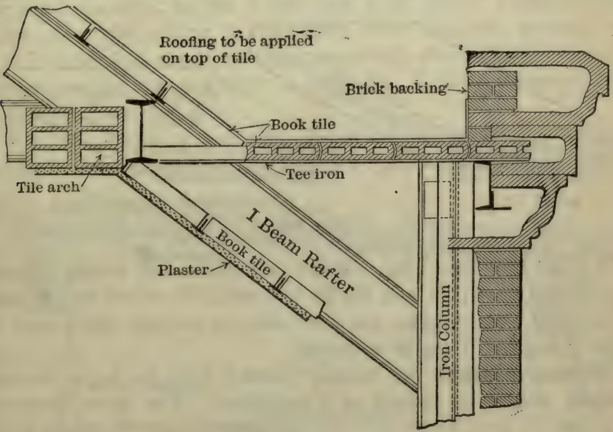


FIG. 64.

The author believes that reinforced cinder concrete, or reinforced porous terra-cotta tile (Johnson System) afford the best and also the most economical construction for fire-proof pitched roofs. Either of these constructions may be filled between or on top of the rafters without the use of purlins, except about once in 6 to 10 feet, to prevent sliding and to stiffen the roof.

"Three-inch plates of concrete with expanded metal imbedded have been successfully used up to spans of 6 to 7 feet and in some cases even to 8 feet.

"The concrete is deposited on wooden centrings, as in the floor construction, and the upper side is smoothed off during the setting and is then floated smooth and straight to receive the roof-covering." * The roof-covering, usually slate, or

* Freitag.

clay tiles, may be nailed directly to the concrete, as cinder concrete holds the nails nearly as well as does wood. (Note.—The above applies only to cinder concrete, as it is quite impossible to nail into rock or gravel concrete.)

With concrete roofs the rafters should also be surrounded with concrete held in place by metal lath. With terra-cotta roofs, the beams should be incased with terra-cotta blocks.

Fig. 65 shows the standard shapes of book tile and solid roofing tile. These are made 2, $2\frac{1}{2}$, and 3 ins. thick, and $15\frac{1}{2}$,

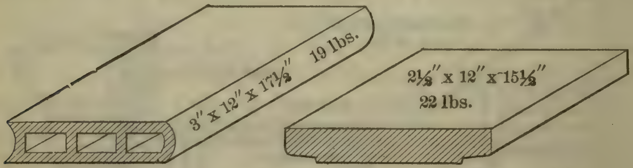


Fig. 65.

$17\frac{1}{2}$, and $23\frac{1}{2}$ ins. long. Three-inch book tile weighs about 13 pounds per sq. ft. and the $2\frac{1}{2}$-inch solid tile about 16 pounds.

Both of these shapes are also used for ceilings and where a light fire-proof filling is required.

Mansard Roofs are usually framed with 4-, 5-, or 6-inch rafters, riveted or bolted to a wall-plate. The space between the rafters may be filled with cinder concrete, hollow partition tile, or blocks extending from rafter to rafter, as in Fig. 66. Slate or tiles may be nailed directly to cinder concrete or to porous terra-cotta.

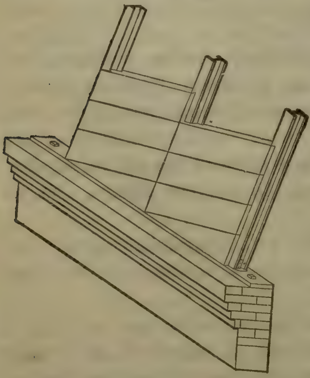


Fig. 66.—Mansard Roof.

Probably the best provision for attaching slate or tiles, however, is to nail $1\frac{1}{4}'' \times 2''$ wood strips to the outer face of the concrete or terra-cotta, the strips being set at the proper distances apart to receive the slate or tile, and then plastering between the strips with cement mortar. This gives a better nailing for the roofing,

and the wood strips would not be affected by fire until the slate was practically destroyed.

With concrete or partition tile filling, the rafters may be spaced 5 to 6 feet apart, while with single blocks, as in Fig. 66, they cannot be spaced more than 2 feet on centres.

Suspended Ceilings.—Office buildings, apartment-houses, etc., having a flat roof, require a ceiling below the roof for appearance in the rooms, and also for heat insulation.

In office buildings the upper ceiling is often framed and constructed similarly to the floors, but with lighter construction. More often the ceiling is suspended from the roof, as this requires much less steel and is consequently much cheaper, while it answers the purpose fully as well—i.e., if the roof-beams are efficiently protected.

Fig. 67 shows a common construction for such ceilings; wrought-iron hangers about $1'' \times \frac{1}{4}''$, split at one end to hook

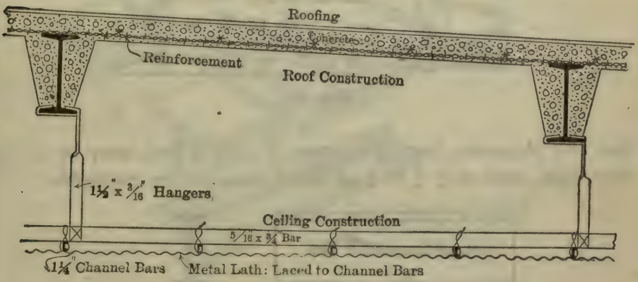


FIG. 67.

over the lower flanges of the roof-beams, are used to support flat steel bars, spaced about 4 feet on centres, and to the underside of these are laced $\frac{3}{4}$-inch or $\frac{7}{8}$-inch channels, 12 or 16 ins. on centres which receive the metal lathing. The bottom of the hangers are bent at right angles to form a seat for the bar, and the bar is laced to the hangers. No bolting or riveting is required, all connections being made by lacing wire, or by bending the iron. Where stiffened wire lath, such as the Roebling or Clinton, is used, the channels may be spaced 16 ins. on centres, but if the ordinary expanded laths are used, it is better to place the channels 12 ins. on centres, and if ordinary lime mortar is used for plastering, a 12-inch spacing is really necessary.

Another system is to use only one set of horizontal bars, which are spaced close enough to receive the lathing, and every bar is supported by hangers. With stiffened wire lathing, and roof-beams spaced not over 5 feet apart, and short hangers, this may be the cheaper system, but without the stiffened lathing, there is no stiffness to the ceiling at right angles to the bars. Where the hangers are 3, 4, or 5 feet long, and the spans between beams are more than 5 feet, the two-bar system, shown by Fig. 67, will require less steel, for the reason

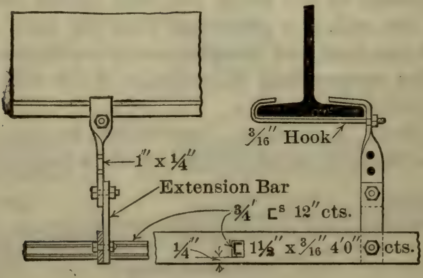


FIG. 68.

that the channels, having a span of only 4 feet, may be made very light, and only $\frac{1}{3}$ or $\frac{1}{4}$ as many hangers are required.

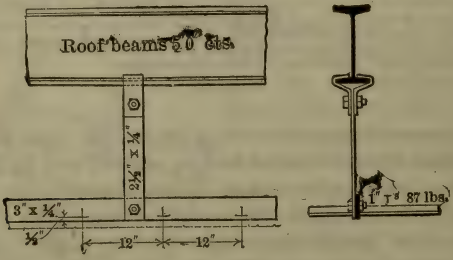


FIG. 69.

In place of the small channels, small tees or flat bars may be used, but where the bars are held by lacing, the channels are to be preferred.

Figs. 68 and 69 (from Freitag's "Fireproofing," p. 286) show very satisfactory details for the construction of the two-bar system.

Instead of the hook shown in Fig. 68, the hanger may be split at the top, and one half bent around one side of the beam flange and the other half bent around the other side.

Where the ceiling is suspended below terra-cotta arches, toggle-bolts are used for the support of the hangers.

The ends of the small bars supporting the lathing are usually spliced by means of sheet-iron clamps, about 6 ins. long, wrapped closely around the bars and hammered tight.

For suspended ceilings under segmental or paneled floor construction, the same methods are employed, except that the hangers are replaced by clips holding the ceiling bars close to the soffits of the beams.

Steel Clips for Fastening Angle- or Tee-bars to I-beams and Channels.—Several years ago Mr. H. A. Streeter,

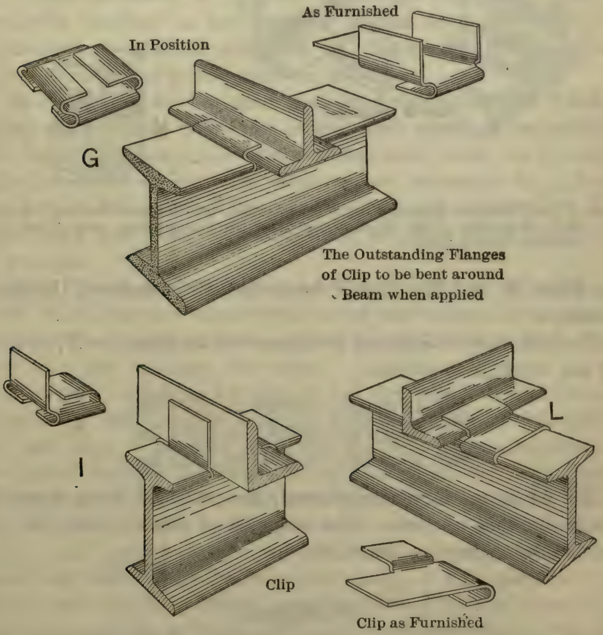


FIG. 70.—Clips for Fastening Tees and Angles to Beams and Channels.

of Chicago, patented a steel clip for connecting angles and tees to I-beams without drilling or bolting, and they have been

extensively used, particularly in roof construction and for suspended ceilings. Besides the saving effected in doing away with the drilling and bolting required by the old method, they also enable the connections to be more quickly made and afford an easy method of adjusting T-bars to any width of tile. Several forms of clips with their application are illustrated

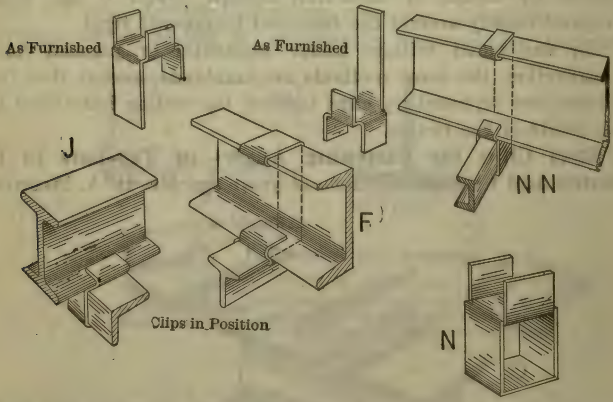


Fig. 71.—Clips for Suspending Tees, Angles, or Channels below I-beams and Channels. Clip N may be used for Suspending any Kind of a Section from a Beam.

by Figs. 70 and 71. Other forms are also made on the same principle.

The safe load which may be supported by clips like N, or NN, $1\frac{1}{2}$ ins. wide, is as follows:

No. 12 gauge = 0.0808 in., 600 lbs.
No. 14 gauge = 0.0641 in., 414 lbs.
No. 16 gauge = 0.0508 in., 215 lbs.

No. 14 gauge is generally used, the material being specially made for this purpose. The strength of the clip may be increased by increasing the width.

Partitions.

Requirement of Fire-proof Partitions.—As a rule the partitions in fire-proof buildings are not required to support any weight, but merely to serve the purpose of dividing the

space into rooms, and to confine a fire to the compartment in which it originates. No greater strength is therefore required in a partition than is necessary to carry its own weight. However, rigidity is required, and that in proportion to its height and unsupported length. Where partitions separate apartments or sections of a story, that is, when they practically are without window or door openings, they should be rigid enough to prevent the passage of water from a hose stream as well as flame. In other cases this may be unnecessary; in fact, at times it may seem desirable that the partitions are easily removed for the purpose of getting at a fire spreading through doors or windows.

The materials of partitions should be incombustible. They should also be poor conductors of heat. It is also desirable that they shall not be affected by water. Lightness is also a desirable quality, as any increase in the dead weight of the construction adds to the cost of the structure. Partitions should also be as sound-proof as possible.

So far as may be, fire-proof partitions should have no window openings in them, and even as few door openings as possible. In many buildings, however, where halls have no openings into streets or courts such windows are necessary for lighting the halls. When this is the case the frames should be made fire-proof and wire glass should be used, the sash being stationary if possible.

Fire Tests on Partitions.—The Bureau of Buildings of New York does not permit the use of any materials or type of construction for partitions in fire-proof buildings that have not met the required fire test.

This test is made as follows: A test structure, 15 feet 3 inches long by 10 feet 3 inches wide, on a 12-inch masonry foundation 2½ feet high, is erected with the material or construction to be tested forming the long sides of the structure. A fire door is provided in one end of the structure for the purpose of charging the fuel. Draft openings are provided in the foundation walls, and four flues are provided, one at each corner. The ends and roof of the structure are made of any material that will stand the fire. Upon a grate, resting on the foundation walls, a wood fire is built and maintained continuously for one hour. The temperatures are read and recorded by means of a standard pyrometer under the supervision of a competent person. A temperature of 1700° Fahr. must be attained at

the end of the first half hour, and maintained during the second half hour. At the end of the hour a stream of water, at 30 pounds pressure through a 1⅛-inch nozzle, is directed against the construction under test, for 2½ minutes. The conditions for acceptance are that at no time during the test shall any fire or water pass through the construction under test, nor shall the construction warp or bulge to any great extent.

Types of Partitions.—Fire-proof partitions that are in common use may be grouped, according to the materials used or the method of construction, as follows:

(*a*) Brick;
(*b*) Terra-cotta;
(*c*) Concrete—stone or cinder;
(*d*) Plaster block;
(*e*) Plaster and Metal.

The choice of construction should be influenced in some degree by the character of the building, and the purposes for which it is used.

Partition Walls.—For bearing partitions (those which support floor beams) there are probably no materials more satisfactory than brick and concrete. The latter may be used either in the form of blocks, or may be poured between forms. Partitions of brick should be at least 12 ins. thick, as thick walls are less affected by heat than thin walls.

Dense tile is also being used for bearing walls with satisfactory results. Some recent tests made at Columbia University show a crushing strength equal to brick. (See p. 732.)

Terra-cotta Partitions.—These are usually built of blocks either square or brick-shaped, according to the particular product used. The square blocks are usually 12″×12″ on the face, and the brick-shaped blocks are usually 12 ins. long but of varying heights. Both shapes are made in thicknesses varying from 3 to 12 ins., the 3-, 4-, and 6-inch blocks being most commonly used; the 4-inch blocks being the most popular for ordinary work. Fig. 72 shows typical shapes of both the square and brick-shaped blocks.

The blocks are commonly set with the voids horizontal, as in Fig. 72, the blocks breaking joint like bricks, but at the ends, and in filling small spaces they are sometimes set vertically.

Fig. 73 shows round- and angle-cornered partition-blocks, which must be set vertically.

"Terra-cotta partitions of a 2-inch thickness have been placed on the market, but have not been extensively used. A 2-inch terra-cotta partition of any strength or efficiency is quite impracticable, and where floor area is so valuable that

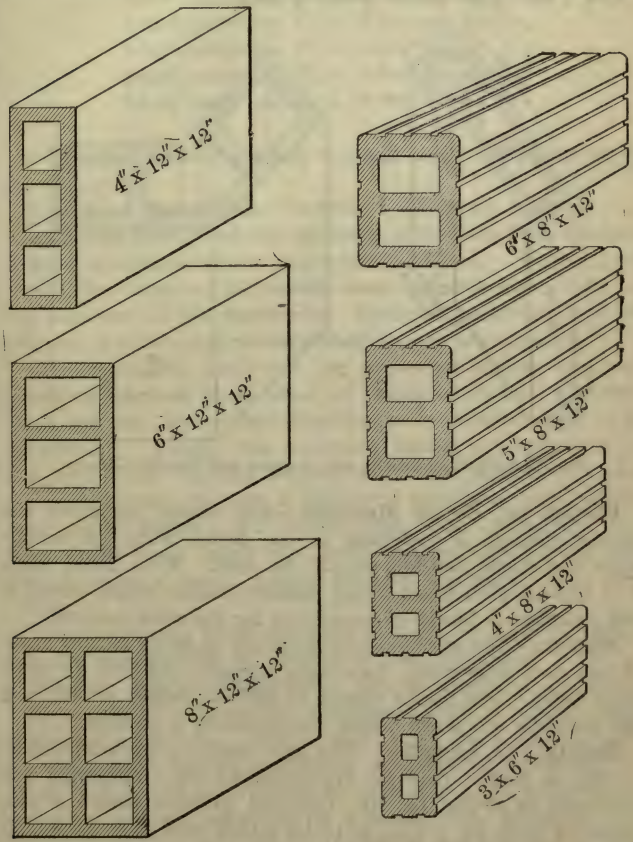


FIG. 72.—Terra-cotta Partition-blocks.

more space cannot be occupied, terra-cotta is not the material to be employed." * Through the addition, however, of band-iron laid between the courses and patented under the style

* Freitag.

"Phœnix," the strength of a 2-inch tile partition is greatly increased.

The "New York" partition (Bevier Patent) consists of 2-inch tile reinforced with truss metal, such as is used in the "New York" floor arch. (See Fig. 33.)

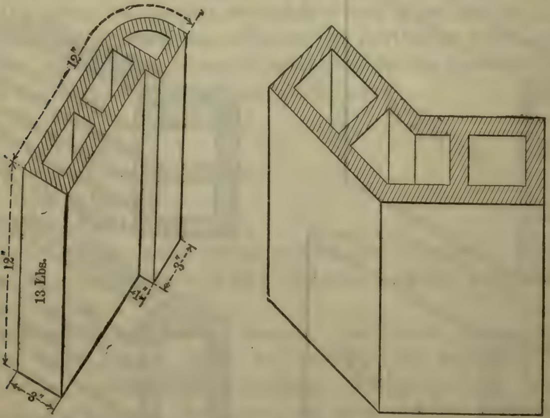


FIG. 73.—Partition-blocks with Angular and Circular Corners.

Porous vs. Dense Material.—For inside partitions the porous material is preferable to the dense, while for outside walls the dense material should be used. With dense tiling it is necessary to insert either wooden nailing strips, which is bad practice, or blocks of porous tile for the same purpose.

Mortar.—Tile partition-blocks should be set in mortar made of one part lime-putty, two parts cement, and two to three parts sand. The blocks *should be well wet* before setting and the partition wet down before the plastering is applied.

Height and Length.—"The safe height of terra-cotta partitions *in inches*, may be approximated by multiplying the thickness in inches by 40. Common practice allows a safe height of 12 ft. for 3-inch, 16 ft. for 4-inch, and 20 ft. for 6-inch partitions. For partitions without side supports the length should not materially exceed the safe height. Doors and high windows may be considered as side supports, provided the studs run from floor to ceiling." *

* Freitag.

Weight.—The weights of either porous or dense terra-cotta partitions will vary as follows:

2-inch partition, 10 to 14 lbs. per sq. ft.;
3-inch partition, 12 to 16 lbs. per sq. ft.;
4-inch partition, 13 to 19 lbs. per sq. ft.;
5-inch partition, 20 to 22 lbs. per sq. ft.;
6-inch partition, 22 to 23 lbs. per sq. ft.;
8-inch partition, 28 to 33 lbs. per sq. ft.;

not including plastering, which will add about 10 lbs. per sq. ft. for both sides.

Concrete Partitions.—Partitions of stone concrete are seldom used because of the necessity of forms in their erection, making a comparatively expensive partition. Unless reinforced they take up too much room. Furthermore they are the heaviest of all partitions. Even in buildings that are entirely of reinforced concrete they are not always used.

Cinder concrete partitions are somewhat lighter and considerably cheaper than those of stone concrete. Yet even these are too heavy and troublesome of construction to be satisfactory. Among the partitions tested and approved by the New York Building Bureau is one that consists of cinder concrete blocks, $2\frac{1}{2}$ and 3 inches in thickness (the thicker ones being hollow), 12 inches high and 18 inches long. They have their edges cast with tongues and grooves that furnish more or less of a bond between the blocks when they are set.

Hollow concrete building-blocks make fairly good partitions, but are objectionable on account of their thickness.

Plaster Block Partitions.—Blocks made of plaster of Paris combined with various substances, such as cinders, wood chips, cocoanut fibre, asbestos, etc., have been used to quite an extent for forming partitions in fire-proof buildings, but while they are to be preferred to partitions built with wooden studding, and will resist fire for a considerable period of time, they cannot be considered as absolutely fire-proof, or suitable for first-class fire-proof buildings. The principal advantage claimed for these partitions is their great lightness and reduced cost as compared with terra-cotta tile. Plaster blocks can be readily cut with a saw, and will receive and hold nails tolerably well.

In the fire tests they have generally shown considerable resistance to the flame and have transmitted less heat than any other form of partition. They did not, however, always

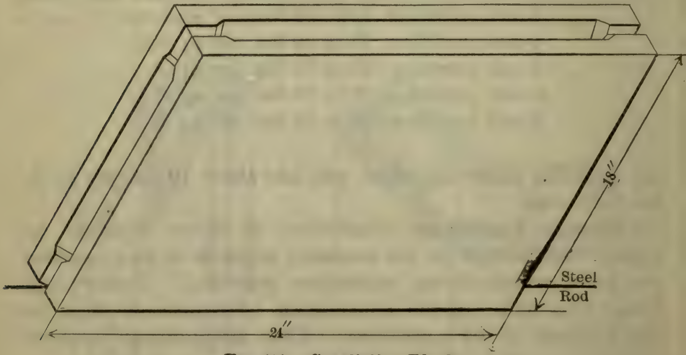


FIG. 74.—Scaglioline Block.

stand the hose stream, some of them being easily pierced, and all of them being more or less washed away by the water.

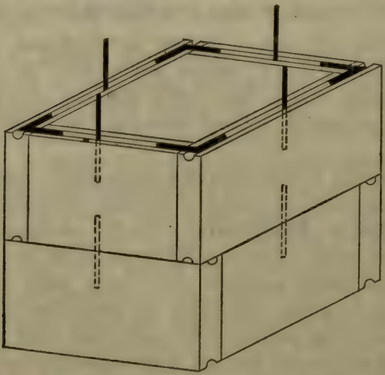


FIG. 75.—Doweled Construction.

They are made of thicknesses varying from 2 to 4 inches, those less than 3 inches thick generally being solid.

The edges of the blocks are generally grooved or otherwise arranged so that the mortar joint forms a key between the

blocks. In the "Scaglioline" * block this keying of the mortar is carried to greater extent than in other blocks, as shown in Fig. 74. The joints are sometimes further reinforced horizontally by rods as indicated in the cut.

In some forms of partitions the blocks are bonded together by means of metal dowels,† running across the horizontal and vertical joints from one block into the adjoining, as shown in Fig. 75. The cut illustrates the use of the block in the construction of dumb-waiter shafts and shows how the blocks are anchored at the corners by iron dowel angles.

The mortar used in laying up plaster blocks is made of the same materials and mixed in the same proportions as that for terra-cotta partitions. (See p. 816.)

All of the partitions in the newer portions of the Monadnock Block, Chicago, and in many other prominent buildings of Chicago and New York are of plaster blocks.

Weight.—Plaster blocks make the lightest practical partition known. The weight of the blocks per square foot may be taken as follows:

Thickness of block, inches . .	2	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	5	6	8
Weight in lbs. per sq. ft.	7	$8\frac{1}{2}$	$9\frac{1}{2}$	$10\frac{1}{2}$	12	15	18	22

The plaster boards, 1 in. thick, weigh 4 lbs. per sq. ft. About 8 lbs. per sq. ft. should be added to the weight of the partition tile to obtain the weight of the partition when plastered both sides.

Mackolite.—The best known and most extensively used of the plaster blocks are the Mackolite Hollow Blocks, made by Mackolite Fireproofing Company of Chicago. Mackolite partition tile are made of the general shape shown by Fig. 76, and of 3, $3\frac{1}{2}$, 4, 6, 8, and 12 ins. in thickness. The 3-, $3\frac{1}{2}$-, and 4-inch tile are made 48 inches long, and the others 30 inches long, all of the tile being 12 inches high. The blocks are laid in regular courses, breaking joint as in cut-stone work. Lime mortar is used for setting. In fitting around openings or at angles the blocks are cut with a saw which effects a material saving in time and material. It is claimed that the blocks make a very strong partition. The composition of the blocks is

* Made and controlled by the Fireproof Partition Co., 1 Madison Ave., N. Y.

† Patented by the Sanitary Fireproofing and Contracting Co., N. Y.

plaster of Paris mixed with certain chemicals, reeds, and fibre. Reeds of the same length as the blocks are placed in the moulds, and the plaster of Paris and fibre is then mixed with water to which the chemical has been added and poured around the reeds so that they shall be nowhere exposed. The reeds give longitudinal strength to the blocks while the fibre makes them tough and elastic. The material sets in about one half hour, after which the blocks are kiln-dried for four days.

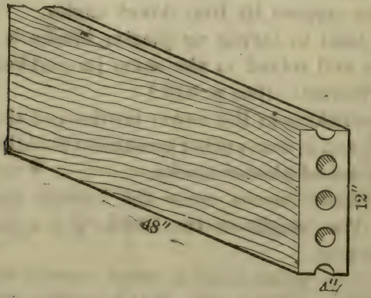


Fig. 76.—Mackolite Partition Tile.

Gypsinite Partitions.—A partition that bids fair to make an economical fire-proof partition has recently been put on the market by the Gypsinite Co., of New York. The main feature of this partition is the stud which is handled and erected in the same manner as a wooden stud in the ordinary non-fire-proof partition. The stud is composed of wood nailing strips completely protected and imbedded in a material known as gypsinite concrete, a plaster composition, not a true concrete. The studs are carefully made and are plumb and true. Metal lath or plaster boards are secured to the studs and plastered, completing the partition, about 4½ inches thick. (See Fig. 77.)

This partition is but slightly heavier than the ordinary partition of wooden construction. It is quite as stiff and strong as a good tile or other partition, and the nailing feature of the studding facilitates the application of wood-trim. It is said to be particularly sound-proof, and the spaces between studs afford an opportunity for the concealment of pipes, wires, etc.

Gypsinite studding in stock size are $3'' \times 3'' \times 12'$, and weigh 3 pounds to the foot. They can be made any size that may be required.

In the partitions the studs are usually placed 16 inches on centres and bridged as may be required. They are fastened to the floor or ceiling by the use of sills and plates of the same material, or light channel irons, which are spiked to the fire-proofing. The manufacturers believe that in large quantities they can be furnished as cheaply as wooden studs and that the partitions can be erected as cheaply as ordinary lath or plaster partitions.

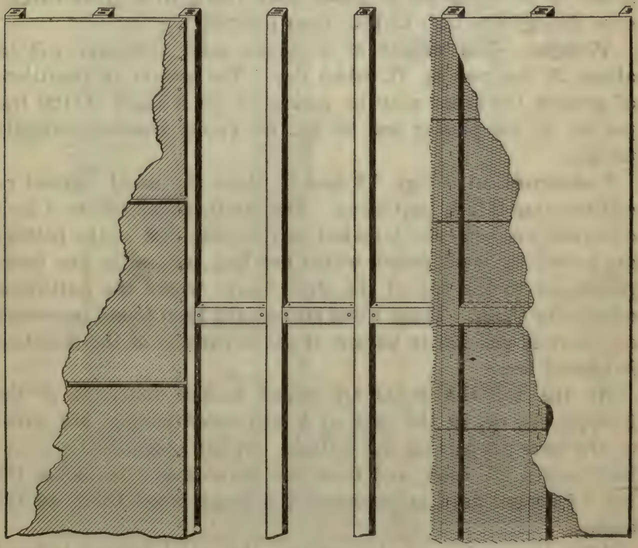


FIG. 77.

Plaster and Metal Partitions.—Thin partitions of plaster applied to metal lath and studding so as to make a solid partition finishing about 2 inches thick, have been very extensively used in fire-proof buildings.

They are remarkably stiff, owing to the adhesion of the plaster to the steel, and they are lighter and occupy less space than any other practical fire-proof partition of equal strength.

In the fire tests these partitions act very much as the plaster-block partitions, resisting thoroughly the passage of the flames. But the plaster always washes off when the hose is applied and the lath becomes exposed. The rigidity of the metal

fabric on the metal studding has been considered by firemen a disadvantage, as it is very difficult to cut through when it becomes necessary to get at a fire.

The construction of the partition is practically the same for the different fabrics used and described on pp. 778 to 784. This lath or fabric appears to be subject to corrosive influences of the plaster. In the demolition of the Pabst Building, New York, the metal lath used throughout in the partitions was found to be corroded, in about four years, to a great extent, even though the lath had all been painted.

Weight.—The weight of a 2-inch solid partition will be about 20 lbs. per sq. ft. when dry. The weight of partitions of greater thickness may be estimated on a basis of 120 lbs. per cu. ft. for plaster and 96 lbs. for cinder concrete, slightly tamped.

Construction.—Figs. 78 and 79 show the usual method of constructing 2-inch partitions. The studs, usually $\frac{7}{8}$- or 1-inch channels, are bent and punched at the ends, and at the bottom are nailed to wood strips, which are first secured to the floor-panels, or to the top of the steel beams where the partitions come over them. These wood strips have been found necessary as a sort of cushion to permit of the expansion of the studding in case of fire.

At the top the studs are nailed to the underside of the floor-panels, or, in the case of a suspended ceiling, are wired to the bars supporting the ceiling. At the openings $1 \times 1 \times \frac{3}{16}$-inch angles are used, and these are bored every 16 inches for No. 12 screws used in attaching the rough wood frame to the angles.

After the studding is in position, the metal lathing is laced to one side of the studding with No. 18 galvanized wire.

After the lathing is in place the carpenter should attach wooden grounds for securing the base, chair-rail, picture-moulding, etc. These are secured by staples and when the partition is plastered become very rigid.

In plastering these partitions, five coats of plaster are required to make a good job: a scratch coat on one side, a brown coat on each side, and the usual white coat on each side for finishing.

It is essential for all thin partitions that a hard-setting mortar be used, such as Acme cement, King's Windsor, Adamant, Rock Wall, and many others.

The partitions acquire thier stiffness largely from the solidity of the plastering, hence the firmer and harder the plastering the more substantial the walls.

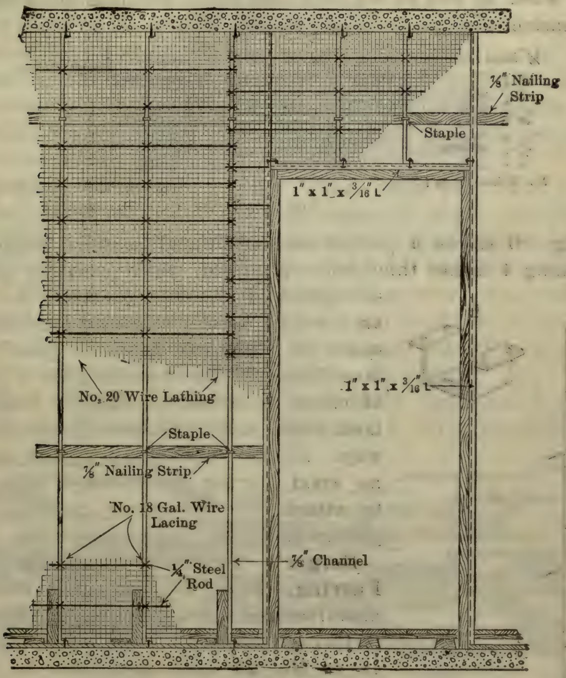


FIG. 78.

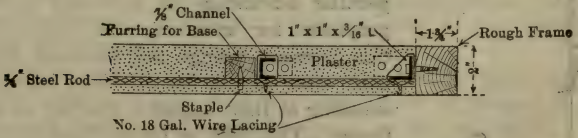


FIG 79.

Double Partitions.—Electric wires and ½-inch gas-pipe can be run in the 2-inch solid partition, but if it is desired to run larger pipes, double partitions, i.e., partitions with lathing on each side of the studding must be used. For these partitions, 2-, 3-, or 4-inch channels or flat bars set edgeways may

be used, sheet steel channels being probably the most economical. When the space between the studding is not filled with mortar or concrete, the double partition will not stand fire and water as well as the solid partition, while it is much more expensive.

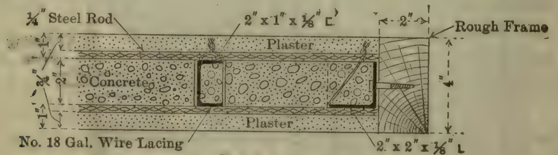


FIG. 80.

Fig. 80 shows a partial section through a solid partition finishing 4 inches thick when plastered, which possesses great strength and absolute resistance to fire and water, besides affording convenient space for pipes and a thicker jamb for door frames. This partition has a core of cinder concrete, with metal lath on both sides, and is plastered in the usual way. As the concrete will receive nails, no wood furring is necessary in order to attach the base-board, chair-rail, or picture-moulding.

Berger's Economy Studding and Furring.—Fig. 81 illustrates a patent stud manufactured by the Berger Manufacturing Company. It is made of No. 18 or No. 20 sheet steel, and in five sizes, varying from ¾ to 1¼ inches. The peculiar advantage of this stud is the provision for attaching the lath. For this purpose prongs are punched from both sides of the flange, which are left standing at right angles to the face of the flange. The lath is placed against the stud, the prongs pressed through the meshes and then turned up over the lath with a hammer, fastening the lath more firmly and securely than by any other method.

FIG. 81.

The ends of the studs are secured by sockets which are

fastened to the floor and ceiling, a clear space being left above the top of the stud to permit of expansion.

Where partitions interesct or angles occur, angle-irons with prongs are used in place of the T.

By using this stud and expanded metal lathing, a saving in cost can be effected over the construction shown by Fig. 13. These T's are also used for supporting suspended ceilings under I-beams, the T's being secured to the flange of the beams by specially designed clips. Furring strips and channels are also made on the same principle.

Spacing of Studding.—For 2-inch solid partitions with $\frac{7}{8}$-inch rolled channels or 1-inch economy studs, the studs should be placed 12 inches on centres when the height of the story exceeds 10 feet. When the height of the story is less than 10 feet, a spacing of 16 inches will answer. For hollow partitions with 2-inch studs, the studs can be spaced 16 inches on centres for story heights of 16 feet and less. For greater heights they should be placed 12 inches on centres.

Truss Metal Lath.* — A remarkably rigid and strong partition, without the use of studding, is being constructed

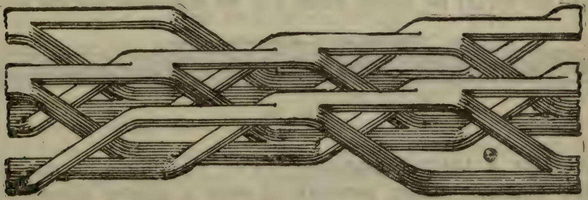


FIG. 82.—Truss Metal Lath (Kuhne Patent).

from truss metal lath. This is an expanded-metal lath of the form shown in Fig. 82. The thickness of the sheets or depth of the truss is about 1 inch.

It is at present manufactured in the form of sheets up to 30 ins. wide and to 9 ft. 4 ins. long. It is at present made from soft steel, black and galvanized, of No. 24, 26, and 28 gauges.

It is much heavier and stiffer than any other expanded-melat form, the No. 28 gauge weighing 0.67 lbs. per square foot, with a cross-section of 0.18 square inch per foot in width, the No. 26 gauge 0.8 lbs. per square foot, with a cross-section of 0.216

* Manufactured by the Truss Metal Lath Co., New York.

square inch per foot in width, and the No. 24 gauge, 1.06 lbs. per square foot, with a cross-section of 0.3 square inch per foot in width.

This lath is said to take mortar better than any other form. A partition of this construction in one of the test structures at Columbia University has passed through and withstood, without any sign of distress, the fire and hose streams of five successive tests.

Triangular Mesh Fabric.—In Figs. 83 and 84 are shown two methods in which the fabric of the American Steel and

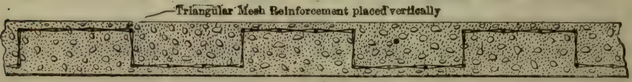


FIG. 83.

Wire Co. (Fig. 45) can be employed in the construction of good, stiff solid partitions. The cross or bond wires of the fabric are so woven to the longitudinal wires that the fabrlc can be placed in the position indicated. Fig. 83 shows how a 6 -inch partition would be made, while Fig. 84 indicates the use of the

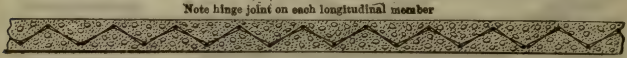


FIG. 84.

fabric for a thinner partition. For the filling-in material, either stone or cinder concrete or even a plaster composition can be used.

Styles of Wire Lathing.—Wire lathing is now made in great variety to meet the requirements of the different plastering compositions and the varying conditions of construction.

Plain lathing is plain wire cloth, usually $2\frac{1}{2} \times 2\frac{1}{2}$ meshes to the inch, made from No. 17 to No. 20 wire. No. 20 is more generally used than any other gauge.

The lathing is also sold plain, painted, and galvanized. Painted or galvanized lathing should be used in connection with special hard-plaster compounds. Painted lathing costs about one cent per square yard more than "bright" lathing.

Galvanizing the wire cloth after it is woven adds very much to its stiffness, as the zinc solders the wires together where they

cross. Galvanized lathing is also less liable to corrosion before the plastering is applied than the plain lathing.

The usual widths of plain lathing are 32 and 36 ins., although the Roebling lath may be obtained of any width up to 8 feet.

All wire lathing should be stretched tight when applied, so as to insure a firm surface for plastering. For this purpose stretchers are supplied by the manufacturers.

Stiffened Wire Lath.—Owing to the difficulty of stretching plain wire cloth tight enough to make a firm foundation for plaster and the necessity for short spacings for the bearings, three varieties of stiffened wire lath have been introduced and extensively used with very satisfactory results. All three varieties are applied so that the stiffening rib will run at right angles to the bearings.

The Clinton stiffened lath has corrugated steel furring strips attached every 8 ins. crosswise of the fabric by means of metal clips. These strips constitute the furring, and the lath is applied directly to the underside of the floor-joist, or to planking, furring, brick walls, etc. This lath is made in 32-in. and 36-in. widths, and comes in 100-yard rolls. The manufacturers of this lath also make a lath stiffened with round rods $\frac{1}{8}$ in. to $\frac{1}{4}$ in. in diameter, spaced from 8 ins. to 12 ins. apart. It can be had either galvanized or japanned, 18 to 22 gauge.

The Roebling standard wire lath is made of plain wire cloth, in which at intervals of $7\frac{1}{2}$ ins. stiffening ribs are woven. These ribs have a V-shaped section and are made of No. 24 sheet-steel $\frac{1}{2}$ and 1 in. in depth. The $\frac{1}{2}$-in. rib is the standard size for lathing on woodwork. This lathing requires no furring, and is applied directly to woodwork or walls with steel nails driven through the bottom of the V, as shown in Fig. 85.

The No. 20 V-rib stiffened lathing affords a satisfactory surface for plastering, when attached to studs or beams spaced 16 ins. apart.

The 1-inch V-rib lathing is used for furring exterior walls. It provides an air space between the wall and plaster.

Where this lath is to be applied to light iron furring a $\frac{3}{16}$- or $\frac{1}{4}$-inch solid steel rod is substituted for the V-rib, and the lathing is attached to light iron furring with lacing wire. This lath is distinguished by the term "solid rib stiffened wire lath."

The Roebling lath, whether plain or stiffened, is made with $2\frac{1}{2}\times2\frac{1}{2}$, 3×3, and $2\frac{1}{2}\times4$ mesh, the latter being known

as "close warp." The $2\frac{1}{2}\times 2\frac{1}{2}$ mesh should be used for ordinary lime and hair mortar, and the 3×3 or $2\frac{1}{2}\times 4$ mesh for hard plasters and thin partitions. This lathing is also sold bright, painted, and galvanized.

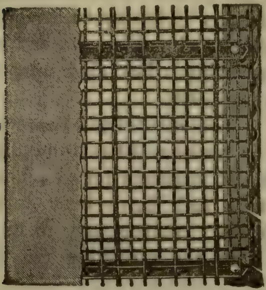


Fig. 85.

The No. 20 painted wire has been extensively used and much of it has been in service for from six to eight years and is now apparently as good and strong as ever, so that there appears to be no necessity in ordinary work of using heavier wire or galvanized netting.

The galvanized wire is stiffer than the painted, and would possible wear longer, but it is doubtful if the advantages are at all proportionate with the cost.

Width.—Wire lath can be furnished to order in any required width up to 10 feet. In widths less than 18 ins. there is a small charge for "stripping." Before ordering, it is very important to ascertain the proper width, especially in stiffened lath, as it is desirable to have the edges of the lath joined at supports when applied to woodwork; and lap at supports when laced to iron furring. When the lath is not of the proper width the results will not be so good and there is liable to be a waste of material.

The standard width of plain and of V-rib stiffened lath is 36 ins. When beams or studs are spaced 16 ins. centre to centre, the lath should be 32 or 48 ins. wide.

Expanded Metal Lath.—The first expanded lath was invented by Mr. John F. Golding, and the patents on the lath and the method of manufacturing are controlled by the Expanded Metal Company.* Very large quantities of this lath have been used for both fire-proof work and for lathing on wooden construction. It is made from strips of thin, soft, and tough steel by a mechanical process which pushes out or expands the metal into oblong meshes, and at the same time reverses the direction of the edges, so that the flat surface of the cut strand is nearly at right angles with the general surface of the sheet.

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* Main office, New York.

For plastering, two styles, "A" and "B" lath, Fig. 86 (0.6×1.5 ins. mesh), and the "Diamond," Fig. 87 (0.41×1.2 ins. mesh, 24 and 26 Stubbs gauge). Both kinds are made in sheets 8 ft. long and from 18 to 24 ins. in width.

This lath being flat and of considerable stiffness, does not require to be stretched, and can be fastened directly to the underside of floor-joist or to wood studding. If used on plank it should be fastened over metal furring strips. When applied to studding the lath should be placed so that the long way of the mesh will be at right angles to the studding, as this insures

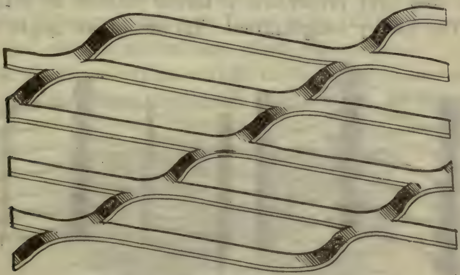

FIG. 86.—"A" and "B" Lath.

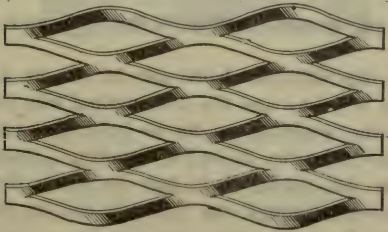

FIG. 87.—Diamond Lath.

the greatest rigidity. The studding or furring strips should be spaced 12 or 16 ins. on centres, and the lathing secured with staples 1 in. long, driven about 5 ins. apart on the stud or joist. The lath, when applied, is a scant $\frac{1}{4}$ in. thick, and to obtain a good wall $\frac{1}{2}$-inch grounds should be used.

Herringbone Lath.—This is another form of expanded metal lath that has been extensively used during the past three or four years. It is made in four grades, *A* and *AA*, *B* and *BB*. The *B* grades are made in wider sheets, and are more open and

consequently not as heavy or stiff as the *A* grades. The general appearance of the *A* and *B* grades is shown by Fig. 88.

The *A* and *AA* grades and the *B* and *BB* grades differ only in that the *AA* and *BB* grades have a smaller mesh and are consequently stiffer than the *A* and *B* grades. The *AA* grade is the stiffest of all and the most expensive, the *A* grade comes next, the *BB* third, and the *B* grade is the lightest of all.

The *A* grade is probably most used. For ceilings "*A* flat" or "*AA* flat" should be specified. The short cross ribs of the "flat" lath are turned after being expanded, diminishing the size of the key and presenting a larger surface to support the plaster. The heavy longitudinal ribs are at an angle of about

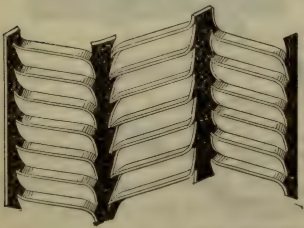

FIG. 88.—Herringbone Lath.

45 degrees to the general surface of the lath and give much stiffness to it.

In applying the lath the sheets should run at right angles to the studding or joists, and the longitudinal ribs should slope down against the studding so as to hold the mortar. For fastening to wood, the No. 12 or 14 "poultry" staple is used. Except for the *AA* grade, it is best to space the studding 12 ins. on centres, although the *A* grade can be used with a 16-inch spacing.

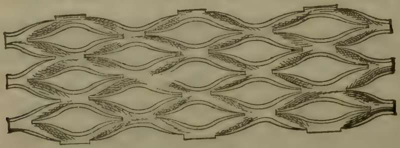

FIG. 89.—Imperial Lath.

Imperial or Spiral Lath.—Fig. 89 shows still another form of expanded-metal lath on the market. This lath is lighter

than most of the laths made from sheet metal, and is also a little cheaper. It is furnished in sheets 48½ ins. long and 16 ins. wide, put up in bundles of 25 sheets. Being so short the sheets nest and pack very closely and are easily handled by one man. Large quantities of this lath have been used, and it seems to be much liked by plasterers.

Perforated Sheet-metal Laths.—There are some six or more styles of metal lath made from sheet iron or steel by perforating the sheets so as to give a clinch to the mortar. The sheets are generally corrugated or ribbed, also, in order to stiffen them and keep them away from the wood. There is not a great difference between these laths, although some styles may possess certain advantages over the others.

In general, those styles which have the greatest amount of preforations, or which approach the nearest to the expanded

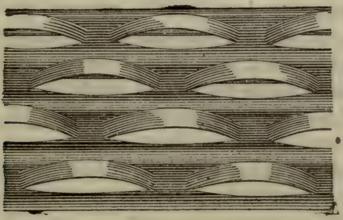

FIG. 90.—Bostwick Lath.

lath, are to be preferred. All of these laths come in flat sheets about 8 ft. long and 15 to 24 ins. in width, and are readily applied to woodwork by means of barbed-wire nails. The nails should be driven every 3 ins. in each bearing, commencing at the centre of the sheet and working toward the ends. These laths work very nicely in forming round corners and coves. Metal lath should never be cut at the angles of a room, but bent to the shape of the angle and continued to the next stud beyond. This strengthens the wall and prevents cracks at the angles.

Of the various forms of sheet-metal lath in common use the Bostwick lath (Fig. 90) is perhaps the best known. It is made of sheet steel, with ribs every $\frac{3}{4}$ of an inch in the width of the sheet, and loops, $\frac{3}{8} \times 1\frac{3}{4}$ ins., punched out between the ribs. It has been extensively used, and is favored by plasterers because it is stiff and easy to apply and requires less plaster than

the more open laths. The lath should be applied with the loop side out.

Sackett's Wall Board.—This is a composite board of alternate layers of plaster and paper, the whole being about $\frac{1}{4}$ inch thick and designed to take the place of either wood or metal lath with some advantages over both.

It is claimed that this board will not warp, buckle, or shrink, and that plastering applied to it will not fall off. As a fire retardent, it is claimed to be equal to metal lath, and when wired to metal studding may be considered as a fire-proof partition. It has the advantage of being very light, and requiring but little plastering material, with a consequent reduction in the amount of water used in plastering.

The boards are 32×36 ins., they may be nailed to wooden studding or flat against solid beams, or plank, and can be cut with a saw.

For plastering the best results are obtained by applying first a brown coat of hard wall-plaster $\frac{1}{4}$ to $\frac{3}{8}$ inch thick; when this is thoroughly set it should be finished with a thin coat of regular hard finish (lime-putty and plaster).

This board has been extensively used in the Eastern States, and in many prominent buildings.

Considering the saving effected in the plastering, the board costs less than metal lath, and but a trifle more than wood laths.

Plaster Boards.—Wall boards, varying in thickness from $\frac{1}{2}$ in. to 2 ins., are made of similar composition to the plaster blocks described on p. 817. These are intended to be used in the same manner and for the same purposes as Sackett's wall board.

Shaft Construction.—The most important partitions in a building are those inclosing interior shafts. Vertical openings through buildings form flues and cause up-drafts. In all buildings, fire-proof as well as non-fire-proof, therefore, they should be inclosed for two reasons: first, to prevent a fire that would find a natural outlet in such openings from spreading to other floors; and secondly, to prevent as far as possible fire getting into these openings where the draft would greatly increase its fury.

To be thoroughly effective the inclosed walls should be of the same construction as the outside walls of the buildings, namely, brick, stone, or concrete. They need not necessarily

be of the same thickness as the outside walls, but 12 ins. is recommended as a minimum. In less important structures terra-cotta partitions are sometimes used as such inclosure walls.

In the walls inclosing elevator shafts no openings except those necessary for entrance doors should be permitted. The doors should be of fire-proof construction (see p. 846) and solid. Glass lights are sometimes provided in such doors, though not good practice, and if so wire glass only should be used in accordance with limitations given on p. 848.

In interior light and vent shafts openings must necessarily be provided, but here again the construction of the window frames and sash and the glazing, should be as far as possible as described on pp. 847 and 848.

Whenever the occupancy of a building admits of it, the staircases should also be inclosed in masonry walls, with fire-proof doors at the openings. Unless so inclosed the staircase forms a flue for the flames, so that the stairs are exposed to intense heat. In such situations, even an absolutely fire-proof stairway could not be used during a fire, and possibly it is for this reason that greater pains have not been taken to make stairs fire-proof.

Shaft walls should in all cases be carried three feet or more above the roof.

Deadening Quality.— The resistance to the passage of sound through fire-proof partitions is an important consideration in buildings used for apartments, and where the rooms are to be used as music studios or for conservatory work, it becomes a matter of great importance.

In Jan., 1895, some tests were made to determine the relative deadening qualities of the different partitions shown by Fig. 91, the object being to decide upon the construction that should be used in Steinway Hall, Chicago.

The rank in sound-proof efficiency of the different partitions tested is shown by the numbers at the right of Fig. 91.

The 4-inch porous partition was used, but was not a success. In the Fine Arts Building, in the same city, double partitions, similar to No. 1, were used, and it is said that they have been a great success.

It is surprising to note that in the tests above mentioned, the 2-inch solid-plaster partition (common mortar) ranked higher than those with double studding. The relative cost

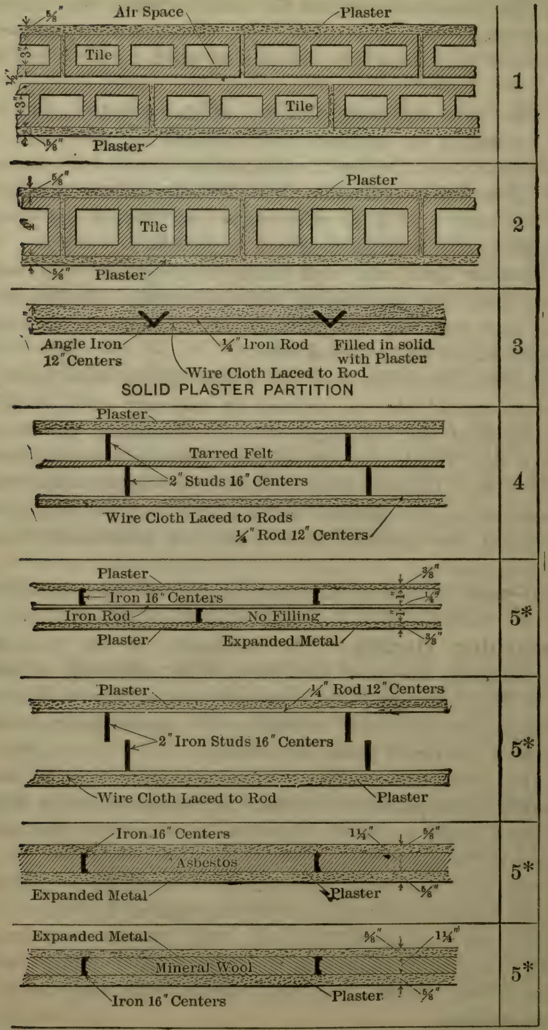

FIG. 91.—Sound carried through probably because of Metal Connections.

of partitions Nos. 1, 2, and 3, including plastering, is given by the Illinois Terra-cotta Lumber Company as \$1.86, \$1.16, and \$1.14 respectively.

In 1892, Prof. Charles L. Norton tested the sound-deadening qualities of several forms of partitions, with the purpose of selecting the best incombustible sound-proof partition for the dormitories of the New England Conservatory of Music, in which practically every room is a music studio. The results of these tests, with a description of the partitions, was published in *Insurance Engineering* for August, 1902.

The various partitions were rated by Prof. Norton as follows:

No.	Room.	Side.	Scale.	Composition.
1	E	Left	100	Cabot's quilt, 3 thick + metal lath.
2	E	Right	95	" " 2 " + " "
3	E	Rear	95	" " 2 " + " "
4	C	Rear	85	Sackett board, 2 felt on [s.
5	C	Left	85	" " 2 " " [.
6	C	Right	80	" " 2 "
7	D	Rear	75	Metal lath + paper.
8	D	Right	75	" " + " + felt.
9	B	Right	60	Two 2-in. Keystone block with 2-in air space.
10	A	Rear	50	4-in. National terra-cotta blocks.
11	B	Rear	50	3-in. Keystone blocks.
12	A	Right	45	3-in. National terra-cotta blocks.
13	B	Left	40	2-in. Keystone blocks.
14	A	Left	40	2-in. National terra-cotta blocks.
15	D	Left	30	2-in. metal lath, solid plaster.

"Nothing more is to be inferred from the numerical efficiencies (under 'scale') than that the first partition is about three times as good as the last, and that the numerical interval between any two partitions on the list merely indicates the order of the magnitude of the difference between the partitions."

Professor Norton recommended a partition of Sackett board and plaster with two thicknesses of Cabot's quilt between the plaster board, and this construction was adopted. The studding was put up the same as for the 2-inch solid partition, the quilt secured to each side of the studs, and the plaster board was wired on to the studding through the quilt. This also makes about as light a partition as it is possible to obtain.

Furring of Outside Walls.—The outside walls of fireproof buildings are generally finished on the inside by plastering applied directly to the masonry. When the walls are of brick, it is often desirable to fur them so that there will be an air space between the plaster and the masonry to prevent the

passage of moisture. This furring should be either of terra-cotta or metal, and never of wood.

For this purpose furring brick are most generally used. They are made of brick clay and of the same size as common brick, but are hollow. They are built up with the rest of the wall, on the inside face, and bonded into the rest of the wall by the usual header courses.

Partition tile are also often used for the inner 4 ins. of brick walls, the tile taking the place of a row of brick, as in Fig. 92.

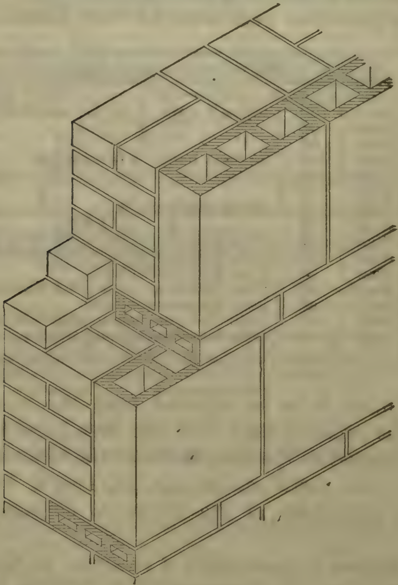

FIG. 92.

By this means dampness is excluded without additional thickness to the walls, and the only additional expense is the extra cost of the hollow tile over the common brick. When using either furring blocks or hollow tile, the mason should be careful not to drop mortar into the hollow spaces.

Where walls are furred or lined with tile, solid porous terra-cotta blocks should be built in wherever nailings are required for base, picture moulding, etc.

In some cities, as in New York, the laws require that when

furring brick or tile is used it shall not be counted as a part of the thickness of the wall. Wire lathing, with a 1-in. V-rib woven in every $7\frac{1}{2}$ ins., also makes a good furring for brick walls, as it is easily applied and affords an air-space between the wall and plaster.

All of these devices also protect the walls from being warped by heat during a fire, and prevent the passage of heat through the walls in summer and winter.

Metal Furring.—To produce architectural forms in the interior decoration of fire-proof buildings, metal furring and lathing are now used almost universally.

The furring is always of a sham nature, and is never employed to carry loads of any magnitude, so that the only requirement is that it shall be incombustible and furnish a satisfactory ground for attaching the metal lath.

For coves, cornices, false beams, etc., the furring members are made of light bars, angles, tees, or channels attached to the walls by means of nails, staples, or toggle bolts, and to the steel beams by means of bolts, hangers, clips, etc. The furring pieces are bent or shaped to the approximate outline of the finished plaster work, so that when the lathing is applied it will not require more than $1\frac{1}{2}$ or 2 ins. of plaster to give the desired outline. For plane surfaces the furring should be brought to within $\frac{3}{4}$ in. of the plaster line. Deep beams, etc., should be braced by diagonal rods, to prevent distortion.

All structural steel members should always be fireproofed back of the furring.

The lathing is secured to the furring by means of No. 18 galvanized lacing wire.

The spacing of the furring should be either 12 or 16 ins., according to the kind of lath that is to be used.

When chases in walls are covered over, it should be done with metal furring and lath. The casings for vertical pipe lines should also be of this construction with the space at the floor level filled solidly about the pipe with fire-proof material, to cut off all connection between stories.

Fire-proof Flooring. —The floor surfaces of most fire-proof buildings consist of hard-wood flooring secured in the usual manner to nailing strips imbedded in the concrete or in the filling above the same. It is sometimes desired to have a flooring that is incombustible. The New York Building Code requires that in all buildings over 150 feet in height,

the floor surfaces shall be of stone, cement, tiling, or similar incombutsible material, or they may be of wood treated by some process to render it fire-proof. For warehouses and factories, floors finished with Portland cement are about as satisfactory as any flooring, and cement floors have been considerably used for the guest rooms of hotels. In the latter rooms, the floor is covered by a carpet, which is secured to wood strips imbedded in the cement around the borders of the room. This makes a very sanitary floor, and is as easy to the feet as a carpeted wood floor.

For public corridors, banks, lobbies, toilet rooms, etc., either encaustic, vitreous, ceramic, or marble tiling is generally used. In France and Germany large quantities of cement tile are used. Cement tiles have also been introduced in this country, but as yet they have not been able to compete with encaustic tile.

In most buildings, however, the use of stone, cement, or tile flooring is undesirable. These materials are cold and trying to the feet. Cement floor surfaces do not generally wear well. Asphaltic flooring is sometimes used, but it is not pleasing in appearance. This material and different floor tiles are discussed on pp. 1448 and 1444 respectively.

The characteristics of fire-proof wood and its availability for this purpose are quite fully covered in the discussion of that material on p. 736.

Composition Flooring. — Several attempts have been made to obtain a flooring material which could be spread over an entire floor without joints, and at the same time be elastic, wear well, and withstand water, acids, etc., and not be too expensive. Various mixtures of magnesite, asbestos, fine sand, sawdust mixed with linseed-oil and some binder like chloride of magnesium, have been put on the market under different names, all more or less meeting the requirements above stated and also being fire-proof.

These materials are shipped in the form of a dry powder to the point where they are to be used, there to be mixed with a specially prepared liquid. The resultant is a plastic material which is laid upon the surface, to be covered much like ordinary cement or plaster. The materials harden in from twelve to twenty-four hours in moderately dry weather, when the floor is ready for use. When properly laid the floor presents a smooth, fine-grained, and continuous surface, resembling linoleum.

These materials are made in various colors, such as red, white, yellow, brown, gray, black, blue, and green, and can be laid on wood, stone, concrete, asphalt, cement, or metals. Another advantage is that they can be carried up on the walls so as to form a coved base, without cracks or joints.

Among such floorings that are on the market and have been used in a number of buildings may be mentioned: Asbestolith, made by the Asbestolith Manufacturing Co. of New York; Lignolith, controlled by the Hecla Iron Works, Brooklyn, N. Y.; Monolith; Crown Sanitary Flooring, made by the Robt. A. Keasbey Co., New York; Alignum; Taylorite, made by Ronald Taylor; New York; and Magnesia Building Lumber, sold by Keasbey & Mattison Co., New York.

Interior Finish.—In buildings in New York City in which the flooring must be of incombustible material, the interior finish, including the doors, door jambs, window frames, sash, base, and trim, must also be of incombustible materials. The same materials that are acceptable as flooring can also be used for this interior finish. Several of the largest buildings in New York, including the Fuller Building, have all the trim of fire-proof wood. In the Hotel Gotham, New York, all the doors and interior finish are made of alignum.

Kalamein.—Besides these materials metal or wood covered with metal may be used. In the Kalamein process, thin sheets of metal are drawn through dies in such a manner that they take the shape of any board or moulding and have the edges pressed into the wood so that the metal covering is held tightly. Copper and zinc are generally used for this purpose.

Metal-covered Door Jambs and Trim are manufactured by the Fire-proof Door Company in a great variety of styles to match their doors. (See p. 846).

Electroplated Trim.—A process recently introduced consists in electrically depositing a layer of copper on the outer surface of wood mouldings or doors. The metallic deposit preserves the markings of the wood grain and makes a very presentable door. A good sample of this work has been installed in the United Engineering Building, New York, by the New York Central Metal Co. of New York.

Some very fine work of this kind has been done by the Hecla Iron Works of New York by electroplating on a fire-proof material known as lignolith.

Cement Trim.—Keene's cement has been used for many years for running base mouldings, door and window trim, etc., and in many European buildings practically all of the interior finish is of this material. Any moulding can be "run" in it with good sharp angles, and it is sufficiently hard to stand ordinary usage.

Fig. 93 shows a door opening with trim of Keene's cement. This detail can be further improved by covering the wooden

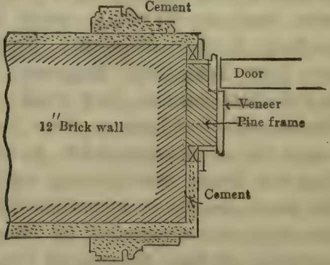

FIG. 93.—Section through Door Jamb.

frame and door with thin metal. The metal and cement can be painted as desired.

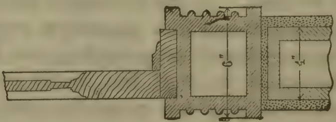

FIG. 94.

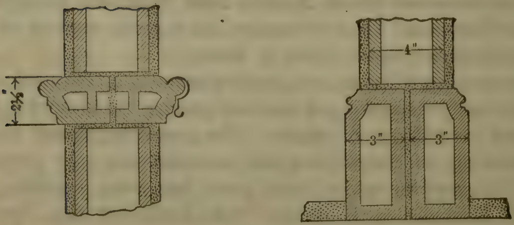

FIG. 95.

Moulded Hollow Tiles are also being substituted for the ordinary wood finish. The "Amelia Apartments," erected by

H. B. Camp at Akron, Ohio, in 1901,* is built almost entirely of hollow tile. "The base, picture-mould, and architraves around doors were made of special formed tile, as shown by Figs. 94 and 95. These tile were afterward painted to harmonize with the color-decoration scheme. All of the floors throughout the building are covered with a cement composition composed of Sandusky cement and ground wood, troweled down smooth and level."

Metallic Furniture and Fittings.—In offices, banks, libraries, and public buildings, the furniture and fixtures are about the only articles on which a fire can feed—if the building itself is fire-proof—and if these are made of incombustible materials there is no chance for a fire to gain headway, or to do much damage. For a number of years The Art Metal Construction Company has been making metallic fixtures from rolled steel plates finely finished in baked enamels relieved by brass and bronze trimmings. Almost anything in the way of furniture and fittings, even to roll-top desks and highly ornamental cabinets, may now be obtained in metal, and many libraries, banks, and court houses have been fitted up and furnished entirely with incombustible cabinet work.

Stairs.—In a majority of fire-proof buildings the architects have contented themselves with putting in incombustible stairs of iron, with perhaps slate or marble treads. As pointed out in the first pages of this chapter, unprotected iron cannot be considered as fire-proof, but it is difficult to protect the iron work of a stairway, as usually built, and at the same time preserve an ornamental effect. If exposed metal construction is to be used, cast iron is much to be preferred to steel, as the cast metal will retain its shape under severe heat far better than thin facings or frameworks of steel.

Slate and marble treads and platforms should never be used in stairs without a support beneath. When subjected to heat, marble and slate will crack and fall away, leaving the stairs impassable. A fire department captain in New York City recently lost his life through the collapse of a marble platform. If these materials are to be used, therefore, there should be a sub-tread of iron or concrete beneath them.

A really fire-proof stair should be constructed with as little iron work as possible, and that incased in fire-resisting materials.

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* Described in *Fireproof*, for July, 1903.

It is possible and practicable to build stairs of clay tiles, brick or reinforced concrete that are absolutely fire-proof. The stairs in the Pension Building at Washington are built of brick with slate treads, and in many of the earlier government buildings the stairs are of stone. Stones suitable for stairs, however, will not stand heat as well as cast iron. Part I. of "Building Construction" * contains descriptions with illustrations of some brick stairs.

The Guastavino Company have built several stairways on their system, using flat clay tile imbedded in cement. No iron work whatever is used in this construction, hence it is eminently fire-proof.

Fig. 96 shows a partial section of a tile stairway used in the Amelia Apartment Building, Akron, Ohio. The blocks were

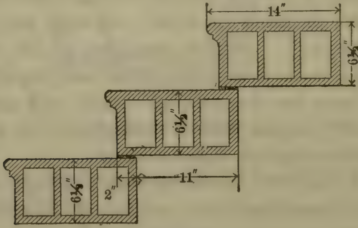

FIG. 96.

of hard-burned material, glazed, and 4 ft. long. They were supported upon the partition walls and were used by the mechanics for carrying up material during the erection of the building.

Reinforced concrete with slate or marble treads also offers a good material for the construction of stairs and permits of very elaborate and complicated construction.†

Fig. 97 ‡ shows the construction of the stairs in the new Government Printing Office at Washington.

These stairs have steel girders and strings which are inclosed in the solid concrete, which is moulded to form the steps and risers, as shown in the detail. The steel strings, however, are hardly necessary, as the reinforcing bars will give sufficient strength.

* By Frank E. Kidder.

† See also page 881*p*, Chap. XXIV.

‡ From the *Engineering Record*, of Dec. 6, 1902.

Some excellent details for ornamental iron stairs were published in the March, 1903, number of *Fireproof,* in an article by Mr. J. K. Freitag.

The corrugated sheet metal, known as Ferro-in-clave (see p. 790) offers a very convenient foundation for cement stairs.

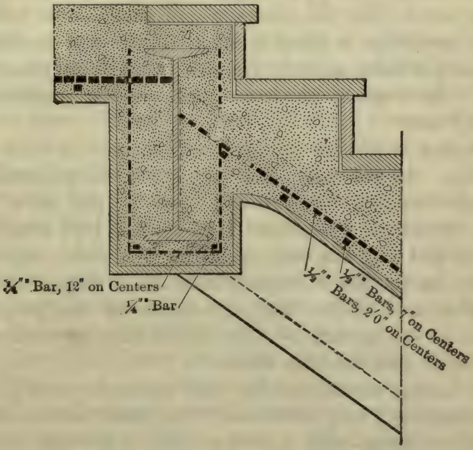

FIG. 97.

when built between walls or partitions or with an open string Fig. 98 shows one way in which the material has been used

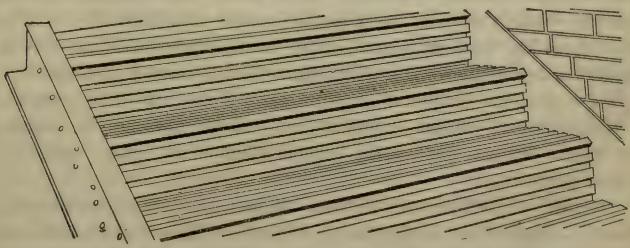

FIG. 98.—Stairs with Treads and Risers of Ferro-in-clave.

the stairs being finished with about 2 ins. of cement over the metal and plastered underneath. The Ferro-in-clave is bolted to lugs or brackets screwed to or cast on the strings. Slate

or marble treads and risers may be bedded in the mortar if desired.

Roof Coverings.—The materials ordinarily used for the roof covering of fire-proof buildings are: 1. Tar and gravel; 2. Asphalt and gravel or sand; 3. Vitrified tiles, brick or slate tiles over tarred felt. Tar and gravel, or asphalt felting and gravel, or sand, offer the cheapest roof suitable for a fire-proof building, and when a good quality of felt and distilled pitch or the best grades of asphalt are used, make a very satisfactory covering. Such roofs, however, require to be renewed about every ten years.

The roofing is put on in the same manner as over wooden construction, the felt being laid directly on the concrete.

Probably the best flat roof that can be put on a building is one of vitrified or slate tiles, laid over five plys of tarred felt. The felt is laid and mopped as for a gravel roof, and the tiles are bedded on the felt in cement mortar. Vitrified tiles, about 8 ins. square and $1\frac{1}{2}$ ins. thick, are made for this purpose, and slate tiles 12 ins. square by 1 in. thick have been used. Flat vitrified brick tiles are also used.

Gravel roofing should not be used on roofs having an inclination exceeding $\frac{3}{4}$ inch in 1 foot. For pitch roofs, either slate, clay tiles, or metal tiles may be used. Clay tiles will stand exposure to fire better than slate, and are to be preferred, especially some of the patent interlocking tiles. (See also pp. 1426 to 1437.)

Window Protection.—To be thoroughly protected against the outside hazard, buildings must have the openings in the outside walls provided with some means of effectively closing that opening against flame. The same provision should be made for openings in the partition walls of large buildings.

Four general types of devices are in use for this purpose: (*a*) tin-covered wood shutters; (*b*) steel shutters or doors; (*c*) metal frames and sash, glazed with wire glass; and (*d*) water curtains.

Merits of Different Types.—When properly constructed the tin-covered wood shutter is still the most effective window protection. "In a very severe fire in Lynn, Mass., in which the heat was intense enough to melt most of the tin from the outside of the tinned plates covering the shutters, it was found afterward that the wood was charred only to a depth of about $\frac{3}{8}$ inch. The shutters were warped slightly but afforded suffi-

cient protection against the heat to allow men to remain behind them to put out such fire as occasionally crept through. This would not have been possible behind iron shutters under similar conditions." *

Steel shutters, under the action of heat, warp very readily and transmit considerable heat. They are the cheapest type of window protection.

"There is one objection to the use of shutters on window openings and that is, that they depend on fallible human agency to be effective. They must necessarily be open while the building is in use. When the need for them comes they are apt to be overlooked and are not closed. Certain it is that on many buildings they are not closed at night." †

The metal frame and wire-glass windows are not unsightly, as most any kind of shutters is apt to be. They are more likely to be closed at night and more readily closed when necessary. They do not hide a fire and are more easily opened when necessary to reach a fire. The one serious objection to them is the intense radiation of heat through the wire glass.

Tin Covered Wood Shutters and Doors.—The effectiveness of this device depends on its construction. "Only well-seasoned non-resinous wood, dressed, tongued and grooved in narrow boards, should be used. Wood containing moisture or resin may generate, under heat, sufficient steam or gas to force off the tin covering and expose the wood to the flame. The body of the door should consist of two or three layers of such boards laid at right angles with each other and fastened together by clinch nails. The best grade of tin should be used. No solder must be used, and the tin plates should be lock-jointed, with the nails in the seams. The nails must be long enough, at least 1½ inches, to secure a good hold beyond the depth to which the wood is likely to char, which is about ¾ inch. Under intense heat the wood is certain to char, but if the nails are long enough to hold the tin up against the wood, and the tin is properly put on so as to keep the air out to prevent burning, the shutter will stand under severe strains."†

The hinges, fastenings, or hangers must be bolted to the door, not nailed or screwed, as the latter would pull out during a fire. If hung on hinges, the hinge-hook should be built into the wall. This door was designed for use in mills, but it has

* *Insurance Engineering*, Dec. 1902, p. 557.

† *Insurance Engineering*, Dec. 1902.

worked so satisfactorily that it is generally adopted wherever a fire-proof door is wanted and the appearance is not objectionable.

Fire-proof shutters are also made in the same way.

The National Board of Fire Underwriters issues complete specifications * for this type of door and shutter which should be closely followed for satisfactory results.

Doors of this type, provided on openings in interior partition walls, are often, and should be wherever possible, hung on inclined tracks so that they will close automatically. Where it is desirable to keep them open ordinarily, an automatic release operated by a fusible link is provided.

Other Metal Covered Wood Doors.—Wood doors covered by the Kalamein process (see p. 839) are sometimes used as fire doors where appearance is a consideration. They are not considered equal to the standard tin-covered wood doors.

Paneled fire-proof doors suitable for offices, hotels, and public buildings are made on the same principle as the standard tin-covered door, stamped sheet steel being used in place of tin.

The Thorp Fire-proof Door Co. of Minneapolis manufacture a very ornamental door having a core of three thicknesses of pine, asbestos covered, and the whole covered with two sheets of steel which are joined on the edges by a patented joint. Panels are sunk by hydraulic pressure. The doors are made with solid and glass panels, and special panelling and moulding may be made to detail. The doors are finished either duplex copper or brass, any finish, solid copper or brass, or painted one coat at factory for finishing at building to match wood.

All hardware is fitted to the doors at the factory without charge. Aside from their fire-proof quality these doors have the advantage that they do not swell or shrink.

Steel Fire Doors and Shutters.—For a satisfactory steel door an $\frac{1}{8}$-inch sheet of steel should be used, reinforced on the back with a frame of angle irons not less than $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$, and increasing in size with the door or shutter.

These doors or shutters may operate in one of three ways: (1) Swinging on hinges, (2) sliding on tracks, or (3) rolling vertically.

The swinging doors or shutters are the most reliable as there

* To be had for the asking.

are no complicated parts to get out of order. They should be hung on eyes built into the masonry walls.

Sliding doors or shutters must have the rails on which they operate protected by metal shields to prevent obstruction.

For larger openings the rolling shutters are generally preferred. They are made in horizontal jointed sectional strips, which wind up on a roller placed in a pocket above the opening, the ends moving in metal grooves to hold them in place.

They generally operate vertically, though some are made to operate horizontally, the rollers being set vertically in pockets at the sides of the openings. These latter are more liable to derangement.

The vertically operated doors or shutters are balanced by springs or weights so as to make them move up or down easily. Where they are intended to be closed only when needed, they are slightly weighted and held open by means of fusible links, so that in case of fire they will close automatically.

Sheet-metal Window Frames and Sash are now made which are weather-tight and perfectly practicable in all respects, and should be used wherever fire-proof windows are desired. The sash are made especially for holding wire glass.

These sheet-metal windows are made in a great variety of forms to meet all purposes, the sashes may be stationary, pivoted either horizontally or vertically, hinged or double hung with weights, like an ordinary window. For factories, warehouses, stairways, and elevator shafts a stationary lower and pivoted upper sash is quite commonly used, as this is the cheapest type of window.

The double-hung windows are now made to work as smoothly as wooden sash in ordinary box frames. For offices, hotels, etc., a window having two sashes, glazed with wire glass, that close and lock automatically in case of fire, and a third inner sash glazed with clear glass, gives all of the advantages of an ordinary window with the additional advantage of fire-protection and a better diffusion of light. Metal fly-screens can also be used with these windows.

All movable sash, glazed with wire glass,* should be provided with a device by which the sash will close and lock automatically in case of fire.

When the contents of the building are inflammable and the

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* See page 738.

exposure severe, two thicknesses of wire glass should be used with a ventilated air-space of at least 1 in. between the lights.

Material.—The frames and sash should be of at least No. 24 gauge galvanized iron, or 18-oz. copper. No. 22 galvanized iron and 20-oz. copper are sometimes used for large frames.

The National Board of Fire Underwriters have adopted rules governing the construction of metal frames and sash for fire-proof windows, which may be obtained from the Mississippi Wire Glass Company, One of these rules provides that "The unsupported surface of the glass shall in no case exceed 24×30 ins." Hence metal sash with a glass opening more than 24 ins. in width or 30 ins. in height must be divided by muntins.

The principal manufacturers of sheet-metal window frames and sash are the Smith-Warren Company of Boston, Mass.; Voigtmann & Co. of Chicago; George Hayes Co. of New York; David Lupton's Sons Co., Philadelphia; and W. H. Mullins Co. of Salem, Ohio, from whom circulars may be obtained giving complete information as to styles and details of construction.

Frames and sash of wood covered with metal are sometimes used, the wood furnishing the strength and the metal the protection. "The contention that wood thus inclosed is liable to dry rot is not confirmed by experience, so far as we are aware, and yet the process of drawing the metal over the wood excludes the air sufficiently to prevent combustion when subjected to heat." The use of such wood covered with metal is recommended, however, only when the better and more expensive form of hollow metal construction cannot be secured."*

Water Curtains.—"The vulnerable portion of buildings generally is the front, where great window openings are desired for purposes of light, and where it is considered objectionable on account of appearance to have shutters or even wire-glass windows. These large window openings afford great opportunities for the spread of fire across streets. The danger of damage is much increased where the fronts, as is very common, are made of unprotected metal work. A notable example, illustrating such danger, was the building of the Manhattan Savings Institution, New York City, which was severely damaged and almost destroyed by a fire in a six-story non-fire-proof building across the street. Such conditions might be overcome to some extent perhaps by the introduction of some

† *Insurance Engineering*, Dec. 1902.

system such as the water curtains that were placed on the Chicago Public Library. This is practically a sprinkler system set along the edge of the cornice of the building, and so arranged as to furnish a thin sheet of water in front of the building. Such a sheet would, however, not extend far before it is turned into spray and thus become practically useless. A similar arrangement placed at each window opening might be more useful, though it is doubtful whether it would be of much value in any severe conflagration." *

Precautionary Measures.—No matter how thoroughly a building may be fire-proofed, if it is filled with combustible goods, as in warehouses, stores, and factories, there is always the possibility of a fire, which if unchecked when first started must necessarily entail a great loss and more or less damage to the building. If a fire is discovered and checked in its incipient stage this loss is avoided. There are now many valuable devices for detecting and checking fires, which should be installed in every warehouse, and which may often be placed with advantage in buildings used for other purposes.

The more important of these are:

Automatic alarms.
Automatic sprinklers.
Standpipes, hose-reels, etc.

Automatic Alarms.—By means of very sensitive thermostats, a rise in temperature of 35 degrees above the normal maximum temperature to be expected in the building will cause an alarm to be sounded.

The Montauk Multiphase Cable Company make a fire-detecting wire or cable which can be used in dwellings and other buildings in place of the ordinary bell wire, and, by judicious distribution and arrangement in elevator- and dumb-waiter shafts, coal and wood cellars, closets, store-rooms, and other unoccupied rooms, may be used to give timely warning of fire originating in any of these places.

This fire-detecting wire consists of two conductors. The central wire or core which forms one side of the circuit has a thick coating or wall of fusible metal; over this is an insulating coating. A number of fine wires are wound over this coating to form a second conductor. The whole is then covered with suitable insulation. When flame or a dangerous degree of heat

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* *Insurance Engineering*, Dec. 1902.

comes into contact with this wire it will establish electrical connection between the two conductors and give a signal on the premises equipped.

This wire was approved by the New York Board of Underwriters, Feb. 20, 1901, and has been quite extensively used in the Eastern States.

Automatic Sprinklers.—"An automatic sprinkler is a device for distributing water by means of a valve which is arranged to open under the action of heat, as from a fire which it is intended to extinguish.

"The distribution of water which results from properly located sprinklers, occurs in the form of a rain of jets or drops, and is sufficient to drench almost any inflammable stock beyond the point of ignition. The distribution is also economical, as the water is more evenly applied than from a nozzle attached to a fire-hose, and the source is directly above the fire.

"Whenever combustible merchandise constitutes the contents of a building, automatic sprinklers are of great value, and in buildings of a height so great as to make the upper stories difficult of access, especially if containing large areas and very combustible contents, sprinklers constitute the best protection obtainable." *

Information pertaining to the installation of automatic sprinklers, their cost, where they may be obtained, etc., may be obtained from the Insurance Engineering Station, Boston, Mass.

Stand-pipes and Hose Reels.—In office-buildings, hotels, and apartment-houses, where sprinkler systems are hardly suitable, stand-pipes with hose reels on each floor and the roof ready for instant use, constitute the best means of quickly controlling a fire. The stand-pipe should be from 2½ to 6 ins. in diameter, according to the size and height of the building, and should be connected with the water supply of the building and provided with Siamese connections at the street level for the fire-department. Check valves should be provided, so that when the fire-department engines are attached their force will be added to that of the buildings, pumps, or to that of the water system.

* J. K. Freitag.

CHAPTER XXIV.

REINFORCED CONCRETE.

By Rudolph P. Miller, C.E.

Definition.—The term reinforced concrete is well defined in the regulations of the New York Bureau of Buildings as "an approved concrete mixture reinforced by steel of any shape, so combined that the steel will take up the tensional stresses and assist in the resistence to shear." *

The regulations use the term "concrete-steel" which, like the similar term, "steel-concrete" has now gone out of general use.

History.—The great value of concrete as a structural material when subjected to compression only, has been recognized for centuries. The use of reinforced concrete, however, as a practicable and commercial form of construction is comparatively recent. It is true that as far back as 1869, Francois Coignet of Paris took out letters patent on a combination of iron and concrete and that even before this, in 1867, the principle of reinforcing concrete with iron had been applied by P. A. J. Monier, a gardener of Paris, to the making of large flowerpots, still the general application to building construction did not occur till about the middle of the last decade of the nineteenth century. In its development from its first use to its application to buildings, it was first applied to bridge construction.

The discussion of the subject in this chapter will be confined to its use in the construction of buildings.

The earliest example of a building of reinforced concrete in this country, and probably in the world, is that erected in 1875 by W. E. Ward, near Porchester, New York, in which "not only all the external and internal walls, cornices and towers were constructed of concrete, but all of the beams and roofs were

* This is also the definition of reinforced concrete in the building laws of Chicago. San Francisco, Buffalo, and St. Louis.

exclusively made of concrete reinforced by light iron beams and rods." *

Erection.—In general outline, a building operation in reinforced concrete consists in the usual preparation of the site by excavation or otherwise, the provision of suitable foundations for walls, columns or other supports, the erection of a series of wooden molds or forms, the placing of the necessary steel reinforcement, the pouring of the concrete and the removal of the forms after the concrete has set sufficiently to sustain itself and the load that may come on it during construction. From the beginning of the erection of the forms the successive steps are progressive, that is, the placing of the steel and pouring of the concrete are going on in the lower sections or stories while the forms are being erected for the upper sections or stories. So that in a large operation the carpenters, the steel-setters and the concreters may all be working at the same time, one set slightly in advance of the others without interference one with the others. These several steps in the operation will be considered in more detail in the paragraphs on "Erection."

Materials.

The materials used in reinforced concrete are concrete and steel. The concrete forms the mass of the construction. Its proper use is to resist compression. While it has some tensile strength the amount is so small and so variable that it should always be neglected.

Steel is used for the reinforcing material as it furnishes the greatest amount of strength at the least expense. Wrought iron could be used, but it is practically unobtainable under present conditions, and, as already intimated, its use is not economical.

Concrete.—The concrete consists of a mixture of cement and some aggregate, in definite proportions with the necessary water to cause the setting of the cement.

Cement.—Portland cement should always be used in reinforced concrete, and it should always be tested before being used. Even in small jobs it is important to know that the cement is strong and sound. In purchasing the cement, the certificate of some reliable testing laboratory should be made

* For a further and more extended history the reader is referred to the larger treatises on this subject and to Mr. Edwin Thacher's article in the Engineering News of March 26, 1903.

one of the conditions of acceptance. Under any circumstances, it is always best to have the testing done at some well established and properly equipped cement testing laboratory. The results of tests in temporary laboratories are often abnormal and may lead to unnecessary controversies with the manufacturers.

To be acceptable, a cement should meet the following requirements as called for in the standard specifications of the American Society for Testing Materials.*

Specific Gravity.—The specific gravity of the cement, thoroughly dried at 100° C., shall be not less than 3.10.

Fineness.—It shall leave by weight a residue of not more than 8 per cent. on a No. 100, and not more than 25 per cent. on a No. 200 sieve.

Time of Setting.—It shall develop initial set in not less than thirty minutes, but must develop hard set in not less than one hour, nor more than ten hours.

Tensile Strength.—The minimum requirements for tensile strength for briquettes one inch square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified:

Neat Cement.

24 hours in moist air	150–200 lbs.
7 days (1 day in moist air 6 days in water)	450–550 "
28 days (1 day in moist air, 27 days in water)	550–650 "

One Part Cement, Three Parts Sand.

7 days (1 day in moist air, 6 days in water)	150–200 "
28 days (1 day in moist air, 27 days in water)	200–300 "

Constancy of Volume.—Pats of neat cement about three inches in diameter, one-half inch thick at the centre, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmos-

* For the complete standard specification see Trans. Am. Soc. for Testing Materials, Vol. IV, p. 109. Copies may be had on request from the Assocn. of Am. Portland Cement Manfrs., Phila.

phere of steam, above boiling water, in a loosely closed vessel for five hours.

These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

Sulphuric Acid and Magnesia.—The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

The test for constancy of volume or soundness is of particular importance for reinforced concrete work. When used in large masses an occasional batch of concrete made with unsound cement may not seriously affect the final result, but in reinforced concrete building operations, where the different members of the structures are comparatively small, the safety of the entire building may be jeopardized by the use of a small amount of unsound cement in some important part, such as a column.

Aggregate.—By the term aggregate is understood the materials mixed with the cement to make the concrete, including the sand. In practically all cases, the sand is a necessary element.

Sand.—"The sand should be clean. One may obtain some idea of its cleanliness by placing it in the palm of one hand and rubbing it with the fingers of the other. If the sand is dirty, it will discolor the palm. If the use of dirty sand is unavoidable, its effect upon the strength of the mortar should be investigated. Preference should be given to sand containing a mixture of coarse and fine grains. Extremely fine sand can be used alone, but it makes a weaker mortar than either coarse sand alone or a mixture of coarse and fine sand."*

Coarser Aggregate.—For the coarser material of the aggregate many materials are used and many others have been suggested. Its selection is generally dependent upon local conditions. If possible gravel or crushed stone should be used. Whatever is used should be a clean hard substance that will secure to the concrete the necessary strength; that is, the crushing strength of this material should be equal to or greater than that of the mortar used, at least at the age of 28 days.

In any case, where no reliable information is to be had on the strength of a concrete made from a given aggregate, careful investigation should be made before such material is used.

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* Taylor & Thompson, "Treatise on Concrete, Plain and Reinforced." John Wiley & Sons, New York.

Gravel.—Gravel, like sand, should be clean. If dirty it should be washed before being used. To get the most satisfactory or uniform results, gravel should be screened and graded and then mixed in definite proportions, as the run of the bank will generally not give uniform results.

Stone.—The most satisfactory stone that can be used is trap rock (under which term are included most of the rocks of igneous origin), because of its toughness and great compressive strength.

The granites as they are commercially known, are considered by some equal in quality to trap rock for the making of concrete. The presence of mica in considerable proportion in some of the so-called granites would seem to make them unsuitable.

Lime stones, if the soft varieties are excepted, make excellent concrete so far as strength is concerned. They would, however, seem to affect the fireproof character of the concrete. (See p. 881*y*.)

The harder and more compact sandstones, too, may be used successfully, but great care must be exercised in their selection. Conglomerate, which is in reality a hard coarse sandstone, should give very satisfactory results.

On account of low crushing strength, slate or shale should not be used in concrete.

Besides the stones thus far mentioned, broken brick, terra cotta, furnace clinker and furnace slag have been suggested.*

In the selection of broken brick or terra cotta care must be taken to get hard burned material. The crushing strength of such material, when well selected, is a little more than that of acceptable concrete, twenty-eight days old. But ordinarily commercial brick or terra cotta will not meet the requirement for a good aggregate, and this material should be used only as a last resort and then only after careful investigation.

Cinders.—Furnace clinkers should be clean and entirely free from combustible matter. The writer knows of no data on the strength of clinker concrete. Cinders are used often where fireproofing is the primary consideration, and no doubt good constructions may be obtained, with extreme care, by the use of clinker or cinder concrete, especially if the material is ground, screened and graded as suggested for gravel. But in general

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* See Sec. 110. Building Code Recommended by National Board of Fire Underwriters.

practice the concrete is not uniform in quality and is unreliable in strength. It will therefore not be considered in this chapter, but is treated of, more at length, in the previous chapter on fireproofing. (See p. 734).

Size of Aggregate.—The size of the aggregate may vary from one-fourth inch to two and one-half inches in largest diametrical dimension, depending on the particular purpose for which it is to be used. Where the mass of concrete is comparatively large the aggregate may run as high as two and one-half inches in size. This may sometimes be the case in foundations and in large piers and thick walls. In columns, girders, beams and slabs, very unsatisfactory results would be obtained if so large a stone were used. For such work no stone or other aggregate should be used larger than would pass a one-inch screen. In important girders and columns, especially when the reinforcing bars are closely spaced, the size should be made even smaller.

The maximum sizes allowed for the aggregate in reinforced concrete in the different cities are given below.

New York, Chicago, St. Louis, Buffalo: stone that will pass a $\frac{3}{4}''$ ring, *i.e.*, "Three-quarter inch stone."

Cleveland: for floors and fireproofing, $\frac{3}{4}''$ stone; for other concrete, 2″ stone.

San Francisco: for floors and fireproofing, 1″ stone; for foundations, 2″ stone.

Proportion.—The proper proportion of the materials entering into the concrete is dependent on the size and character of the material. In cities where there are regulations governing reinforced concrete construction the mixture to be used is generally specified. In the absence of other considerations the most satisfactory and reliable mixture is one part of Portland cement, two parts sand and four parts stone or gravel. It is the mixture that has been used in most of the experimental work on reinforced concrete, and there is therefore much trustworthy information to be had.

In the case of large or important operations, however, great economy can often be effected by a preliminary study of the materials to be used and their proper proportions. In general, the most economical mixture is also the strongest, for given materials.

The old method of determining the proportions of concrete by measuring the voids in the coarser particles by means of water

poured into a box containing one cubic foot of the material and then providing that quantity of finer material, assuming the cement the same as sand, is not to be recommended. It does not give accurate or satisfactory results.

A better method is to take the materials to be used and make trial mixtures by varying the proportions, but always using the same amount of cement and water. Place these trial mixtures successively in a measuring vessel of fixed size, tamping the same and noting the height to which the vessel is filled for each mixture. The proportions that give the lowest height, or result in the smallest volume, will give the most satisfactory concrete.

The best and most scientific method, however, is that known as the Mechanical Analysis, devised by Mr. Wm. B. Fuller. In this method the available materials, including the cement, are separated into the various sizes by means of a series of sieves; curves are plotted which indicate the percentages of the whole mass, which pass the several sieves; from a study of these curves the proportions of the different aggregates are determined. For a detailed description of this method the reader is referred to the chapter on "Proportioning Concrete" in Taylor & Thompson's "Treatise on Concrete, Plain and Reinforced."

As an example of the saving possible, the following case, given in the work just referred to, will be of interest:

"The ordinary mixture for water-tight concrete is about $1:2\frac{1}{2}:4\frac{1}{2}$, which requires 1.37 barrels of cement per cubic yard of concrete. By carefully grading the materials by methods of mechanical analysis the writer has obtained water-tight work with a mixture of about 1:3:7, thus using only 1.01 barrels of cement per cubic yard of concrete. This saving of 0.36 barrels is equivalent, with Portland cement at $1.60 per barrel, to $0.58 per cubic yard of concrete. The added cost of labor for proportioning and mixing the concrete because of the use of five grades of aggregate instead of two was about $0.15 per cubic yard, thus effecting a net saving of $0.43 per cubic yard. On a piece of work involving, say, 20,000 cubic yards of concrete such a saving would amount to $8600.00, an amount well worth considerable study and effort on the part of those in responsible charge."

In the ordinances or regulations governing reinforced concrete of various cities the proportions to be used are generally prescribed.

In New York and St. Louis "the concrete must be mixed in the proportions of one of cement, two of sand and four of stone

or gravel; or the proportions may be such that the resistance of the concrete to crushing shall not be less than 2,000 lbs. per square inch after hardening for twenty-eight days. The tests to determine this value must be made under the direction of the Superintendent of Buildings."

In Cleveland no mixture is specified, but the strength to be developed is provided for as for New York.

In Boston and San Francisco the proportion is given as one of cement to six of aggregate.

In Buffalo a 1:2:5 mixture is required.

In Chicago a 1:3:5 mixture is called for.

All these cities require, however, that the strength developed in 28 days shall be at least 2,000 lbs. per square inch.

Strength.—For reinforced concrete work no mixture should be used that does not develop a strength of at least 2,000 lbs. per square inch at the age of twenty-eight days.

The crushing strength of various concretes is shown in the following table;

TABLE I.

COMPRESSIVE STRENGTH OF PORTLAND CEMENT CONCRETE OF DIFFERENT PROPORTIONS.

Proportion			Age months	Compressive Strength per sq. in.	AUTHORITY.
Cement	Sand	Stone			
1	1	0	4	4370	James E. Howard–Tests at Watertown Arsenal
1	2	0	4	2506	" "
1	3	0	4	1812	" "
1	4	0	4	830	" "
1	5	0	4	532	" "
1	6	0	4	169	" "
1	7	0	4	118	" "
1	2	4	4	2178	" "
1	3	6	4	1815	" "
1	4	8	4	1135	" "
1	5	10	4	707	" "
1	6	12	4	738	" "
1	2	2	4	1768	" "
1	2	3	4	1911	" "
1	2	4	4	2147	" "
1	2	5	4	2452	" "
1	2	6	4	2124	" "
1	2	7	4	1650	" "
1	2	8	4	1295	" "
1	2	4	1	2399	G. A. Kimball–Tests of Metals U. S. A.
1	2½	5	1	3255	Taylor & Thompson–Tests at Watertown Arsenal.
1	3	5	1	2042	Watertown Arsenal–Tests of Metals U. S. A.

Working Stresses.—Some formulæ for the strength of reinforced concrete construction provide for the use of the ultimate strength of the concrete and the application of a factor of safety. This practice is not to be recommended as it necessitates either the test of the concrete or the assumption of an ultimate stress. While it is undoubtedly desirable that the concrete should be tested, this is generally impracticable when the building is being designed. It should be done during construction and is done on the best work, to make sure that the concrete is up to the requirements.

Various factors of safety from two and one-half to ten have been proposed. Different factors of safety are used for different members of a structure or for different conditions. This furnishes another reason why it would be better to use working stresses as against ultimate stresses.

The following working stresses are recommended for concrete that will develop a crushing strength of 2,000 lbs. per square inch at twenty-eight days:

Extreme fibre stress in compression.........	500 lbs. per sq. in.
Shearing stress........................	75 " " " "
Direct Compression	350 " " " "

The table on the next page gives the stresses allowed by various building ordinances and other authorities.

Steel Reinforcement.—The function of the steel reinforcement is to take up the tensional stresses, to assist in the resistance to shear and in some cases, as in columns and in beams reinforced at the top, to give additional compressive strength.

Mild or High Steel.—Two grades of steel are used for the reinforcement, mild steel and high carbon steel.

Mild or medium steel is used for all structural shapes and is the ordinary merchant steel. It has an ultimate tensile strength of 60,000 to 70,000 lbs., and its elastic limit is about one-half of the ultimate strength.

High carbon steel has a greater percentage of carbon, and is therefore more brittle. Its ultimate strength is about 105,000 lbs. per square inch, and its elastic limit about 55,000 lbs.

The use of high carbon steel would permit greater stresses in the reinforcement, and consequently a less amount of steel and a greater economy in construction. On account of its greater brittleness, however, it is liable to sudden failures under stress. It is also often found to be cracked or broken when sent to the

TABLE II.

WORKING STRESSES.

Authority.	Extreme Fibre Stress in Comp. in concrete per sq. in.	Shearing Stress in Concrete per sq. in.	Direct Comp. in Concrete per sq. in.	Adhesion of Steel to Concrete per sq. in.	Tensile Stress in Steel per sq. in.	Shearing Stress in Steel per sq. in.	Ratio of Modulus of Steel to Modulus of Concrete (r)
New York....	500	50	350	50	16000	10000	12
Chicago.......	500	75	350	75⅓	Elastic Limit	10000	12
St. Louis......	500	50	350	50	.....	.....	18
Boston.......	500	60	416 Mx 347 Mn	..	.....	.....	15 in Beams and slabs 10 in Col's
Cleveland.....	500	50	350	50	16000	10000	15
Buffalo.......	500	50	350	50	16000	10000	12
San Francisco.	500	75	450	75	⅓ Elastic Limit	10000	15
National Board of Fire Underwriters	500	50	350	50	.....	.....	18

work, and, unless very carefully inspected there is great liability of defective material getting into the structure. Furthermore, much of the so-called high carbon steel in practice has been found upon test to fall far short of the specifications. Its use is therefore to be avoided, unless special care is taken to secure an absolutely reliable article and to have it inspected and tested. For large important work this would be desirable. Ordinarily, however, mild steel should be used, as commercially it is manufactured and sold under such standard conditions that it is reliable. As the modulus of elasticity of high carbon steel is practically the same as that of medium steel, the deformation under any given loading is the same and there is no special advantage in the use of one over the other.

Working Stresses.—The generally accepted working stresses for medium steel are 16,000 pounds per square inch in tension and 10,000 pounds per square inch in shear. Tests have shown that in cases where the failure of reinforced concrete

beams is due to the failure of the reinforcement, the stress in the metal had not more than reached the yield point. This point is somewhat lower than the elastic limit. The working stress in the steel, therefore, should be a fixed proportion of the yield point or the elastic limit. It is held by some that this ratio should not be as high as one to two, but more nearly one to three, reducing the working stress in mild steel as given above to 10,000 or 12,000 pounds per square inch. In using high carbon steel they would advocate a similar ratio of the elastic limit, whatever that may be according to test. Ordinarily 20,000 pounds per square inch is taken as the working stress for high carbon steel.

Allowable working stresses in steel reinforcement in various cities are given in Table II on p. 860.

Tension Members.—Reinforcement is used in a variety of shapes and combinations, nearly all of them patented and some of them forming the basis for so-called systems.

Where the reinforcement is employed to take up tension, as in a beam or girder, the bond between the concrete and the steel is relied upon to develop the tensional stresses in the steel.

The plain bars depend entirely upon the adhesion of the steel and the concrete for the action of the two materials in combination, or the full tensile strength of the rod is developed by anchoring the rods into the concrete at the ends, in which case the beam becomes more analogous to a trussed beam with the rod as the tension member. In cross-section, plain bars are usually round or square, though sometimes flat bars, angles, tees, or other shapes are used. In the use of square bars and some other shapes, it is contended that the edges start initial cracks in the concrete while shrinking during setting.

Deformed Bars.—In the deformed bars the adhesion of the concrete to the steel is supplemented by a mechanical bond due to the shape of the bar. Among them the following are such as have been and are being widely used.

Ransome Bar.—The Ransome Twisted Bars (Fig. 1), are made of square bars twisted cold. The work on the bars in the twisting process increases the tensile strength of the steel. On this account the users of this bar generally assume a working stress of 20,000 pounds per square inch. The patent on this bar has expired and its use is now open to anyone, although because of special machinery and facilities the Ransome Machinery Company can probably furnish the bars at less cost than they could

otherwise be obtained. Strictly speaking, this is not a deformed bar.

These bars can be obtained in all sizes, varying by one eighth

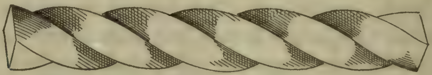


Fig. 1.

of an inch, from ¼ to 1¼ inch. Larger sizes can also be obtained on special order.

Thacher Bar.—The Thacher Bulb Bar (Fig. 2), also patented, has had extensive use, but more particularly in bridges

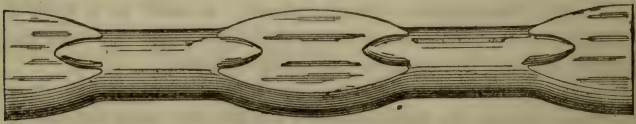


Fig. 2.

and arches. Its particular features need not be discussed further as the patentees have practically replaced it by their new Diamond Bar.

Johnson Bar.—The Johnson Corrugated Bar (patented) (Fig. 3), made by the St. Louis Expanded Metal Company is also said to be made of high carbon steel with an elastic limit of about 58,000 pounds per square inch. In this bar the mechanical bond

Fig. 3.—Johnson Bar.

is effected by a series of corrugations on the sides of a square rod, alternating in position and lapping so that the effective area remains the same for the full length of the bar. The shoulders formed by the corrugations have an inclination with the axis of the bar, such that there cannot be any sliding of the concrete on the same under the force tending to pull the steel through the concrete.

DATA ON JOHNSON BAR.

Nominal Size of Bar.	Net Section.	Weight per Foot.
¼″	0.06	0.24 lbs.
⅓″	0.11	0.38 lbs.
⅜″	0.14	0.48 lbs.
½″	0.25	0.85 lbs.
⅝″	0.39	1.33 lbs.
¾″	0.56	1.91 lbs.
⅞″	0.77	2.60 lbs.
1″	1.00	3.40 lbs.
1¼″	1.56	5.31 lbs.

Diamond Bar.—The Diamond Bar **(Fig. 4)**, recently put on the market by the Concrete Steel Engineering Company, is claimed as the only deformed bar of absolutely uniform section. There is consequently no waste of metal due to the deformations

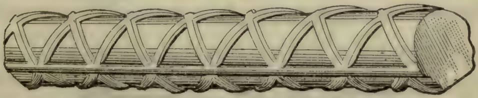


Fig. 4.—Diamond Bar.

as in other bars. The bar is practically a round bar and sudden transitions from one section to another are avoided so that all tendency to produce initial cracks in the concrete is overcome.

The weights and areas of Diamond bars are equal to those of plain square bars of like denominations. Bars may be obtained from the Concrete Steel Engineering Company, of New York, from ¼ inch up to 1¼ inches.

Cold Twisted Lug Bar.—This bar is manufactured by the General Fireproofing Company, Youngstown, Ohio, and, as its name implies, is a twisted bar having small projections or lugs at intervals. It is claimed for this bar that the lugs effectually

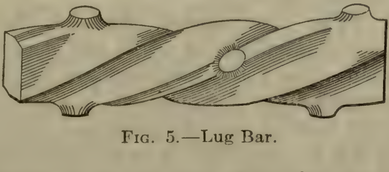


Fig. 5.—Lug Bar.

resist any tendency on the part of the bar to untwist under tension, and that the twisting increases the ultimate strength and raises the elastic limit, at the same time reducing the elongation. (Fig. 5.)

Cold twisted lug bars are rolled in sections corresponding in sectional area to $\frac{1}{4}''$, $\frac{1}{2}''$, $\frac{5}{8}''$, $\frac{3}{4}''$, $\frac{7}{8}''$, $1''$, $1\frac{1}{8}''$ and $1\frac{1}{4}''$ square bars.

DATA ON COLD TWISTED LUG BAR.

Size.	Approx. Wt. per Foot.	Net Sec. Area.	Working Load at 20000 Pounds.
$\frac{1}{4}''$	.222	.0625	1250
$\frac{3}{8}''$	.492	.1406	2810
$\frac{1}{2}''$	.870	.2500	5000
$\frac{5}{8}''$	1.350	.3906	7810
$\frac{3}{4}''$	1.940	.5625	11250
$\frac{7}{8}''$	2.640	.7656	15300
1 $''$	3.450	1.0000	20000
$1\frac{1}{8}''$	4.350	1.2656	25200
$1\frac{1}{4}''$	5.370	1.5625	31250
$1\frac{1}{2}''$	7.700	2.2500	45000

Cup Bar.—The cup bar (Fig. 6), as implied by the name, is provided with a series of cups, which, being filled with concrete, resist the tendency to pull out. It is claimed for this bar that the

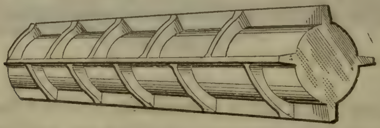


Fig. 6.—Cup Bar.

cups are scientifically designed so that the bar will not slip through the concrete nor shear the concrete along the line of the bar. This bar is manufactured by the Trussed Concrete Steel Company, Detroit, Michigan.

Other types of patented reinforcing bars which may be mentioned are the following:

De Man Undulated Bar.—A bar having an approximately uniform cross section, with undulating faces which are intended to increase the bond.

Universal Type Corrugated Bar.—A flat corrugated bar having rounded undulating edges and depressions rolled in the flat faces. This bar is manufactured by the Expanded Metal and Corrugated Bar Company, St. Louis, Mo.

The Kahn and Golding bars are plain bars, which because of other features will be considered in more detail further on.

Different methods of anchoring the tension bars in reinforced concrete have been used. In the Hennebique System of construction, where plain bars are used, the ends of the rods are split and flared out. (See Fig. 7.) In other constructions the ends of

the bars are simply turned at right angles in such direction as is most suitable. In some instances nuts and washers have been placed at the ends of reinforcing rods. Where reinforced concrete

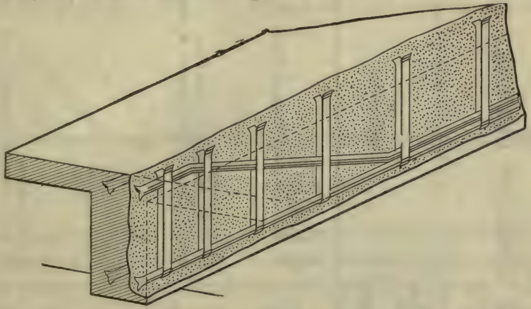


Fig. 7.—Hennebique System.

floors are used in connection with steel columns the rods are run through the web plates or through angle brackets and secured with nuts.

Adhesion.—The bond between concrete and steel for various forms of bars and differing conditions is shown in the table on the next page.

Shear Members.—In many of the tests on full sized concrete beams failure occurred by the development of diagonal breaks near the supports. These are considered by some as the result of the horizontal and vertical shear which is greater toward the supports. More recently they are supposed to be due, in part at least, "to internal tension caused by a stretching and slipping of the rods employed in the reinforcement." *

Stirrups.—On the assumption that they are due to shear, it has been attempted to overcome the defect by placing stirrups in the concrete either vertically or diagonally. In the older constructions they are placed loosely, making only an attempt to keep them in their relative positions in the beams.

In view of the second cause mentioned, however, the tendency of the reinforcement to slip in the concrete should be minimized by one or more of the following methods: (1) By securing the greatest possible adhesion of the steel to the concrete; (2) by attaching the stirrups firmly to the tension rods; (3) by providing some form of anchoring at ends; (4) by mechanical bond.

Kahn Bar.—In the Kahn Trussed Bar the attachment of the stirrups to the tension member is positively secured. The

* Lewis J. Johnson in Journal of Am. Eng. Soc., June, 1904, p. 308

TABLE III.

SHOWING RESULTS OF VARIOUS TESTS ON ADHESION BETWEEN CONCRETE AND STEEL.

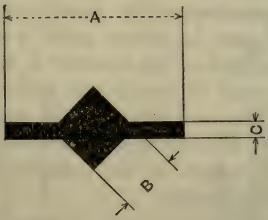
Kind of Bar.	Size Tested.	Concrete.	Age.	Ultimate Strength developed in lbs. per sq. in. of surface in contact.	Authority.
Round	$\frac{1}{2}''$	1:2:4	60 days	412	A. N. Talbot
Square	$\frac{3}{4}''$	1:3:6	30 "	274	C. M. Spofford
Square (Rusted)	$\frac{3}{4}''$		30 "	437	N. Y. C. Rapid Transit Com.
Square (Rusted	$\frac{3}{4}''$		90 "	642	"
Square	$\frac{7}{8}''$		90 "	431	"
Square	$\frac{7}{8}''$		30 "	294	"
Twisted (Ransome)	$\frac{5}{8}''$	1:2:4	31 "	648	T. L. Condron
Twisted	$\frac{3}{4}''$		25 "	500	N. Y. C. Rapid Transit Com.
Twisted	$\frac{3}{4}''$	Neat Cement	7 Mos.	1290	Tests of Metals, Watertown Arsenal(1904)
Twisted	$\frac{3}{4}''$	1:1	"	1318	"
Twisted	$\frac{3}{4}''$	1:2	"	1199	"
Twisted	$\frac{3}{4}''$	1:3	"	701	"
Twisted	$\frac{3}{4}''$	1:4	"	796	"
Corrugated	$\frac{3}{4}''$	Neat Cement	"	962	"
Corrugated	$\frac{3}{4}''$	1:1	"	977	"
Corrugated	$\frac{3}{4}''$	1:2	"	934	"
Corrugated	$\frac{3}{4}''$	1:3	"	735	"
Corrugated	$\frac{3}{4}''$	1:4	"	564	"
Corrugated	$\frac{3}{4}''$	1:2:4	31	640	T. L. Condron
Thatcher	$\frac{7}{8}''$		30	646	N. Y. C. Rapid Transit Com.

bar is a square bar with webs rolled on the same at two diagonally opposite edges. The stirrups are formed by shearing these webs for part of their length along the bar and turning them up, as shown in the cut. (Fig. 8.) These stirrups may be placed so as

FIG. 8.—Kahn Bar,

to turn up in pairs or so as to alternate on opposite sides of the bar, making the spacing of the stirrups closer than when turned up in pairs. Another advantage incidental to the use of this bar is that the greater effective cross section in the steel is at the centre, the point of greatest bending moment. Two disadvantages, however, are the separation of the concrete by the wings above and below the bar, and the limitation as to the effective stirrup length in deep beams. This bar is controlled by the Trussed Concrete Steel Company of Detroit.

The Kahn Trussed Bar can be obtained in the following sizes:



Size.	*A*	*B*	*C*	Weight per foot.	Area sq. in.
$1\frac{1}{2}''$x $\frac{1}{2}''$	$1\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{8}$	1.4	0.38
$2\frac{3}{16}''$x $\frac{3}{4}''$	$2\frac{3}{16}$	$\frac{3}{4}$	$\frac{3}{16}$	2.7	0.78
$3''$x1 $''$	3	1	$\frac{1}{4}$	4.8	1.42
$3\frac{3}{4}''$x$1\frac{1}{4}''$	$3\frac{3}{4}$	$1\frac{1}{4}$	$\frac{1}{4}$	6.9	2.00

Golding Bar.—The Golding Bar (Fig. 9), the product of the Monolith Steel Company of Washington, D. C., is a plain bar of the cross section shown in the cut. The grooves in the sides serve not only to increase the area of contact between steel and concrete for a given area of cross section, but form a means

of attachment for the stirrups as indicated. The grasp of the metal on the stirrups, which is accomplished by machinery, is so great that the stirrup will rupture before a separation will take

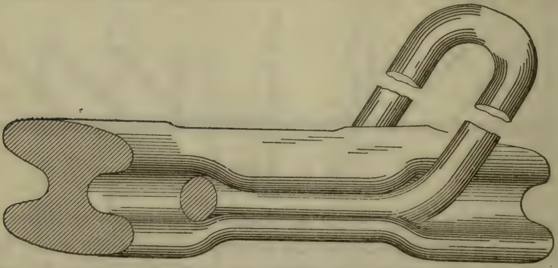


Fig. 9.—Golding Bar.

place, as shown by actual test. Stirrups may be placed at any point and as close together as desired.

This bar may be obtained in the following sizes:

Size..........	$1\frac{1}{2}''$	$1''$	$8/10''$	$\frac{1}{2}''$
Area.........	$2\frac{1}{4}$	1	64/100	$\frac{1}{4}$
Web Member..	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{4}$	3/16

Steel in Compression.—The steel reinforcement in reinforced concrete is used in certain cases to assist in developing compressive strength when the concrete is not sufficient for the purpose, as in the case of beams and girders with rods placed above the neutral axis, and columns with rods placed vertically. The use of the steel reinforcement in resisting compression will be treated more at length in the paragraph on the design of beams, girders and columns.

On account of the uncertainty, however, of the steel and concrete each receiving its proportionate share of the load, the use of steel in compression should be avoided as much as possible.

Position of the Reinforcement.—The importance of the exact position of the reinforcement in the concrete will become more apparent in the discussion of the design of beams (p. 872). It will appear that a slight displacement of the steel will materially affect the construction. If the steel shifts upward the beam is weakened, if it shift downward the protection of the steel against rust or fire is reduced. In the so-called "Unit" systems the reinforcement, including the tension rods and stirrups, are so tied and framed together that after being placed

n the forms the possibility of shifting their positions with respect to the other surfaces of the beam or to one another is practically entirely removed.

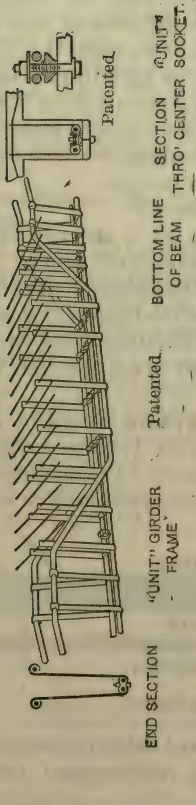


Fig. 10.

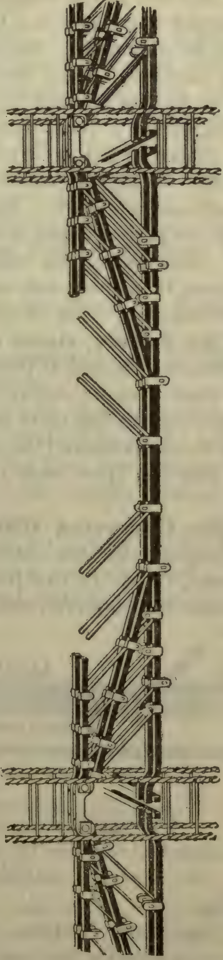


Fig. 11.—Pin-connected Girder Frame.

Unit System.—The particular advantages in the use of a unit system of reinforcement is, as already indicated, the assur-

ance that each and every part of the reinforcement is in its exact relative position, and maintains that position during the placing of the concrete. The reinforcement for each beam or girder is as carefully laid out as the location of cover plate, stiffeners, connection angles and rivets in a built up steel girder. It can consequently be thoroughly inspected and checked before being placed in position. Being marked, its exact location is easily determined by the foreman on the job, from the erection plan. After being placed a quick inspection will show at once whether correctly placed or not, as it must fit and extend the full length of the mold. Being fabricated "off the job" there is less interference between workmen. The fabrication can be done while the molds are getting ready so that there can be more speed in erection. The frames are readily transported and less liable to get mixed than loose rods sent to the job.

The following are types of unit systems:

The Unit System, shown in Fig. 10, is the pioneer of this type of construction and at present is manufactured by the Unit Concrete Steel Frame Company of Philadelphia. Its particular features are the bending up of some of the longitudinal reinforcement near the supports and the use of flat strap U shaped stirrups, punched near the upper ends through which the slab rods are threaded.

The Pin Connected Girder Frame shown in Fig. 11, is manufactured by the General Fire Proofing Company of Youngstown, Ohio. In this frame some of the reinforcement is bent up near the supports, and this reinforcement in adjoining

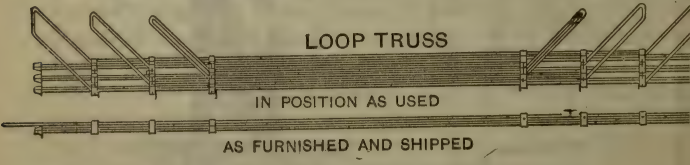


Fig. 12.—Cummings System.

girders is fastened together by means of links and pins forming a tie over the supports making the reinforcement practically continuous.

The Cummings System, shown in Fig. 12, is manufactured by the Electric Welding Company, Pittsburg, Pa. The particular feature of this system is the inverted U shaped stirrups which are shipped flat with the longitudinal reinforcement, but

are bent up to an inclined position on the work. The rods are held together by means of a patented chair.

Design.

Girders, Beams, and Slabs. — Different formulæ have been devised by different investigators based on various theoretical considerations. The formulæ here given have been widely accepted and are offered as simple in form and giving satisfactory results. If anything they err on the side of safety, and furthermore have been found to give results closely in accord with actual tests. They have been adopted by the Prussian Minister of Public Works (see Regulations of 1904), are used by the New York Building Bureau and are accepted by other authorities.

Assumptions.—The formulæ are based on the following assumptions:

(*a*) The bond between the concrete and steel is sufficient to make the two materials act together.

(*b*) A plane section before bending remains a plane section after bending, and the stress and strain in any fiber are directly proportionate to the distance of that fiber from the neutral axis.

(*c*) The modulus of elasticity of the concrete remains constant within the assumed working stresses.

(*d*) The tensional stress is taken entirely by the steel; that is the tensile strength of the concrete is not considered.

Fig. 13 represents a longitudinal section of a reinforced concrete beam under load.

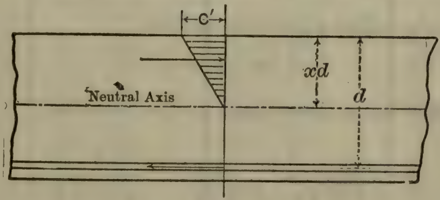


FIG. 13.

The fibers above the neutral axis are in compression and according to our assumptions the strains vary in direct proportion to the distance from the neutral axis, so that the area of compression, representing the total compressive force, is indicated by the shaded triangle.

The total tensional force is concentrated at the center of gravity of the steel reinforcement, and in order that there shall be equilibrium the resultant compressive force and the total tensional force must be equal.

Formulæ for Beams.—From these conditions the following formulæ are derived, in which

S = the allowable tension or working stress in the steel.
C = the allowable unit compression or working stress in the extreme fibre of the concrete.
r = the ratio of the modulus of elasticity of steel to the modulus of elasticity of the concrete.
d = the effective depth of the beam, that is the distance from the center of gravity of the steel reinforcement to the extreme fibre in compression.
x = the ratio of depth of the neutral axis from the extreme fibre in compression, to the effective depth of the beam, so that
xd = the distance of the neutral axis from the extreme fibre in compression.
b = the width of the beam.
p = the ratio of cross section of the steel to the cross section of the beam, considering the beam all of that part of the concrete above the center of gravity of the steel.
M = the moment of resistance of the beam, which must of course be equal to or greater than the moment of the external forces acting on the beam.
K = a factor used for simplification of the formula. This factor is constant for any given steel and concrete.

For beams of rectangular cross section

$$M = Kbd^2 \qquad (a)$$

the value of K being determined by the following:

$$K = S\left(\frac{1}{\frac{2S}{C}\left(1+\frac{S}{Cr}\right)}\right)\left(1-\frac{1}{3\left(1+\frac{S}{Cr}\right)}\right) \qquad (b)$$

which formula can be deduced from the laws of flexure and the assumptions noted above.

In the use of this formula for the value of K it must be remembered that the ratio of S to C, for any given ratio of steel to concrete. p. is a constant so that corresponding values of S and C

must be used. This ratio p, often spoken of as the percentage of reinforcement, is the expression in the first bracket

$$p = \frac{1}{2\left(\frac{S}{C}\right)\left(1+\frac{S}{Cr}\right)} \qquad (c)$$

The value of x is derived from the expression

$$x = rp\left(\sqrt{1+\frac{2}{rp}}-1\right) \qquad (d)$$

Values for K and x for corresponding values of p, for different conditions fixed by the building authorities of different cities are given in Tables IV and V.

Table IV.

X = Coefficient, which multiplied by d gives position of Neutral Axis.
K = Coefficient for determining Resisting Moment. $M = Kbd^2$
C = Extreme fibre stress in concrete.
S = Extreme fibre stress in steel.

$$r = \frac{Es}{Ec} = 12$$

p	x	K	C	S	K	C	S	K	C	S
.0005	.104	5.8	115	12000	6.8	135	14000	7.7	154	16000
.0010	.143	11.4	168	"	13.3	196	"	15.2	224	"
0015	.172	17.0	209	"	19.8	244	"	22.6	279	"
.0020	.196	22.4	245	"	26.2	286	"	29.9	327	"
.0025	.217	27.8	276	"	32.5	323	"	37.1	369	"
.0030	.235	33.2	306	"	38.7	357	"	44.2	409	"
.0035	.251	38.5	335	"	44.9	390	"	51.3	446	"
.0040	.266	43.8	361	"	51.1	421	"	58.4	481	"
.0045	.279	49.0	387	"	57.1	452	"	63.3	500	15500
.0050	.291	54.2	412	"	63.2	481	"	65.7	"	14550
.0055	.303	59.3	436	"	68.1	500	13773	68.1	"	13773
.0060	.314	64.4	459	"	70.3	"	13083	70.3	"	13083
.0065	.325	69.6	480	"	72.5	"	12500	72.5	"	12500
.0070	.334	74.2	500	11929	74.2	"	11929	74.2	"	11929
.0075	.344	76.2	"	11467	76.2	"	11467	76.2	"	11467
.0080	.353	77.9	"	11030	77.9	"	11030	77.9	"	11030
.0085	.361	79.4	"	10618	79.4	"	10618	79.4	"	10618
.0090	.369	80.9	"	10250	80.9	"	10250	80.9	"	10250
0095	.377	82.4	"	9921	82.4	"	9921	82.4	"	9921
.0100	.384	83.7	"	9600	83.7	"	9600	83.7	"	9600
.0105	.392	85.2	"	9333	85.2	"	9333	85.2	"	9333
.0110	.399	86.5	"	9068	86.5	"	9068	86.5	"	9068

TABLE IV.—*Continued.*

p	*x*	*K*	*C*	*S*	*K*	*C*	*S*	*K*	*C*	*S*
.0115	.405	87.6	500	8804	87.6	500	8804	87.6	500	8840
.0120	.412	88.9	"	8583	88.9	"	8583	88.9	"	8583
.0125	.418	90.0	"	8360	90.0	"	8360	90.0	"	8360
.0130	.424	91.0	"	8154	91.0	"	8154	91.0	"	8154
.0135	.430	92.1	"	7963	92.1	"	7963	92.1	"	7963
.0140	.436	93.2	"	7786	93.2	"	7786	93.2	"	7786
.0145	.441	94.0	"	7603	94.0	"	7603	94.0	"	7603
.0150	.446	94.9	"	7433	94.9	"	7433	94.9	"	7433
.0155	.452	96.0	"	7290	96.0	"	7290	96.0	"	7290
.0160	.457	96.8	"	7141	96.8	"	7141	96.8	"	7141
.0165	.462	97.7	"	7000	97.7	"	7000	97.7	"	7000
.0170	.467	98.6	"	6868	98.6	"	6868	98.6	"	6868
.0175	.471	99.3	"	6729	99.3	"	6729	99.3	"	6729
.0180	.475	99.9	"	6597	99.9	"	6597	99.9	"	6597
.0185	.480	100.8	"	6486	100.8	"	6486	100.8	"	6486
.0190	.485	101.7	"	6382	101.7	"	6382	101.7	"	6382
.0195	.489	102.3	"	6269	102.3	"	6269	102.3	"	6269
.0200	.493	103.0	"	6163	103.0	"	6163	103.0	"	6163
.0005	.104	8.7	173	18000	9.7	192	20000	10.6	212	22000
.0010	.143	17.1	252	"	19.0	280		20.9	308	"
.0015	.172	25.4	314		28.3	349	"	31.1	384	"
.0020	.196	33.6	367	"	37.4	408	"	41.1	449	"
.0025	.217	41.8	415	"	46.4	461	"	50.3	500	21700
.0030	.235	49.8	460	"	54.2	500	19583	54.2	"	19583
.0035	.251	57.5	500	17929	57.5	"	17929	57.5	"	17929
.0040	.266	60.6	"	16625	60.6	"	16625	60.6	"	16625
.0045	.279	63.3	"	15500	63.3	"	15500	63.3	"	15500
.0050	.291	65.7	"	14550	65.7	"	14550	65.7	"	14550
.0055	.303	68.1	"	13773	68.1	"	13773	68.1	"	13773
.0060	.314	70.3	"	13083	70.3	"	13083	70.3	"	13083
.0065	.325	72.5	"	12500	72.5	"	12500	72.5	"	12500
.0070	.334	74.2	"	11929	74.2	"	11929	74.2	"	11929
.0075	.344	76.2	"	11467	76.2	"	11467	76.2	"	11467
.0080	.353	77.9	"	11030	77.9	"	11030	77.9	"	11030
.0085	.361	79.4	"	10618	79.4	"	10618	79.4	"	10618
.0090	.369	80.9	"	10250	80.9	"	10250	80.9	"	10250
.0095	.377	82.4	"	9921	82.4	"	9921	82.4	"	9921
.0100	.384	83.7	"	9600	83.7	"	9600	83.7	"	9600
.0105	.392	85.2	"	9333	85.2	"	9333	85.2	"	9333
.0110	.399	86.5	"	9068	86.5	"	9068	86.5	"	9068
.0115	.405	87.6	"	8804	87.6	"	8804	87.6	"	8804
.0120	.412	88.9	"	8583	88.9	"	8583	88.9	"	8583
.0125	.418	90.0	"	8360	90.0	"	8360	90.0	"	8360
.0130	.424	91.0	"	8154	91.0	"	8154	91.0	"	8154
.0135	.430	92.1	"	7963	92.1	"	7963	92.1	"	7963
.0140	.436	93.2	"	7786	93.2	"	7786	93.2	"	7786
.0145	.441	94.0	"	7603	94.0	"	7603	94.0	"	7603
.0150	.446	94.9	"	7433	94.9	"	7433	94.9	"	7433
.0155	.452	96.0	"	7290	96.0	"	7290	96.0	"	7290
.0160	.457	96.8	"	7141	96.8	"	7141	96.8	"	7141
.0165	.462	97.7	"	7000	97.7	"	7000	97.7	"	7000
.0170	.467	98.6	"	6868	98.6	"	6868	98.6	"	6868
.0175	.471	99.3	"	6729	99.3	"	6729	99.3	"	6729
.0180	.475	99.9	"	6597	99.9	"	6597	99.9	"	6597
.0185	.480	100.8	"	6486	100.8	"	6486	100.8	"	6486
.0190	.485	101.7	"	6382	101.7	"	6382	101.7	"	6382
.0195	.489	102.3	"	6269	102.3	"	6269	102.3	"	6269
.0200	.493	103.0	"	6163	103.0	"	6163	103.0	"	6163

TABLE V.

$$r=\frac{Es}{Ec}=15$$

p	x	K	C	S	K	C	S	K	C	S
.0005	.115	5.8	104	12000	6.7	122	14000	7.7	139	16000
.0010	.159	11.4	151	"	13.3	176	"	15.2	201	"
.0015	.191	16.9	188	"	19.7	220	"	22.5	251	"
.0020	.217	22.3	221	"	26.0	258	"	29.7	295	"
.0025	.239	27.6	251	"	32.2	293	"	36.8	335	"
.0030	.258	32.9	279	"	38.4	326	"	43.9	372	"
.0035	.276	38.1	304	"	44.5	355	"	50.9	406	"
.0040	.292	43.3	329	"	50.6	384	"	57.8	438	"
.0045	.306	48.5	353	"	56.6	412	"	64.7	471	"
.0050	.320	53.6	375	"	62.6	438	"	71.5	500	"
.0055	.332	58.7	398	"	68.5	464	"	73.8	"	15091
.0060	.344	63.8	419	"	74.4	488	"	76.2	"	14333
.0065	.355	68.8	439	"	78.3	500	13654	78.3	"	13654
.0070	.365	73.8	460	"	80.1	"	13036	80.1	"	13036
.0075	.375	78.8	480	"	82.0	"	12500	82.0	"	12500
.0080	.384	83.7	500	"	83.7	"	12000	83.7	"	12000
.0085	.393	85.4	"	11559	85.4	"	11559	85.4	"	11559
.0090	.402	87.0	"	11167	87.0	"	11167	87.0	"	11167
.0095	.410	88.5	"	10789	88.5	"	10789	88.5	"	10789
.0100	.418	89.9	"	10450	89.9	"	10450	89.9	"	10450
.0105	.425	91.2	"	10119	91.2	"	10119	91.2	"	10119
.0110	.433	92.6	"	9841	92.6	"	9841	92.6	"	9841
.0115	.440	93.9	"	9565	93.9	"	9565	93.9	"	9565
.0120	.446	94.9	"	9292	94.9	"	9292	94.9	"	9292
.0125	.453	96.2	"	9060	96.2	"	9060	96.2	"	9060
.0130	.459	97.2	"	8827	97.2	"	8827	97.2	"	8827
.0135	.465	98.2	"	8611	98.2	"	8611	98.2	"	8611
.0140	.471	99.3	"	8411	99.3	"	8411	99.3	"	8411
.0145	.477	100.3	"	8224	100.3	"	8224	100.3	"	8224
.0150	.483	101.3	"	8050	101.3	"	8050	101.3	"	8050
.0155	.488	102.2	"	7871	102.2	"	7871	102.2	"	7871
.0160	.493	103.0	"	7703	103.0	"	7703	103.0	"	7703
.0165	.498	103.8	"	7545	103.8	"	7545	103.8	"	7545
.0170	.503	104.6	"	7397	104.6	"	7397	104.6	"	7397
.0175	.508	105.5	"	7257	105.5	"	7257	105.5	"	7257
.0180	.513	106.3	"	7125	106.3	"	7125	106.3	"	7125
.0185	.518	107.2	"	7000	107.2	"	7000	107.2	"	7000
.0190	.522	107.8	"	6868	107.8	"	6868	107.8	"	6868
.0195	.527	108.6	"	6756	108.6	"	6756	108.6	"	6756
.0200	.531	109.3	"	6638	109.3	"	6638	109.3	"	6638

TABLE V.—*Continued.*

p	x	K	C	S	K	C	S	K	C	S
.0005	.115	8.7	157	18000	9.6	174	20000	10.6	191	22000
.0010	.159	17.1	226	"	18.9	252	"	20.8	277	"
.0015	.191	25.3	283	"	28.1	314	"	30.9	346	"
.0020	.217	33.4	332	"	37.1	369	"	40.8	406	"
.0025	.239	41.4	377	"	46.0	418	"	50.6	460	"
.0030	.258	49.3	419	"	54.8	465	"	58.9	500	21500
.0035	.276	57.2	457	"	62.7	500	19714	62.7	"	19714
.0040	.292	64.0	493	"	65.9	"	18250	65.9	"	18250
.0045	.306	68.7	500	17000	68.7	"	17000	68.7	"	17000
.0050	.320	71.5	"	16000	71.5	"	16000	71.5	"	16000
.0055	.332	73.8	"	15091	73.8	"	15091	73.8	"	15091
.0060	.344	76.2	"	14333	76.2	"	14333	76.2	"	14333
.0065	.355	78.3	"	13654	78.3	"	13654	78.3	"	13654
.0070	.365	80.1	"	13036	80.1	"	13036	80.1	"	13036
.0075	.375	82.0	"	12500	82.0	"	12500	82.0	"	12500
.0080	.384	83.7	"	12000	83.7	"	12000	83.7	"	12000
.0085	.393	85.4	"	11559	85.4	"	11559	85.4	"	11559
.0090	.402	87.0	"	11167	87.0	"	11167	87.0	"	11167
.0095	.410	88.5	"	10789	88.5	"	10789	88.5	"	10789
.0100	.418	89.9	"	10450	89.9	"	10450	89.9	"	10450
.0105	.425	91.2	"	10119	91.2	"	10119	91.2	"	10119
.0110	.433	92.6	"	9841	92.6	"	9841	92.6	"	9841
.0115	.440	93.9	"	9565	93.9	"	9565	93.9	"	9565
.0120	.446	94.9	"	9292	94.9	"	9292	94.9	"	9292
.0125	.453	96.2	"	9060	96.2	"	9060	96.2	"	9060
.0130	.459	97.2	"	8827	97.2	"	8827	97.2	"	8827
.0135	.465	98.2	"	8611	98.2	"	8611	98.2	"	8611
.0140	.471	99.3	"	8411	99.3	"	8411	99.3	"	8411
.0145	.477	100.3	"	8224	100.3	"	8224	100.3	"	8224
.0150	.483	101.3	"	8050	101.3	"	8050	101.3	"	8050
.0155	.488	102.2	"	7871	102.2	"	7871	102.2	"	7871
.0160	.493	103.0	"	7703	103.0	"	7703	103.0	"	7703
.0165	.498	103.8	"	7545	103.8	"	7545	103.8	"	7545
.1070	.503	104.6	"	7397	104.6	"	7397	104.6	"	7397
.0175	.508	105.5	"	7257	105.5	"	7257	105.5	"	7257
.0180	.513	106.3	"	7125	106.3	"	7125	106.3	"	7125
.0185	.518	107.2	"	7000	107.2	"	7000	107.2	"	7000
.0190	.522	107.8	"	6868	107.8	"	6868	107.8	"	6868
.0195	.527	108.6	"	6756	108.6	"	6756	108.6	"	6756
.0200	.531	109.3	"	6638	109.3	"	6638	109.3	"	6638

A table of value of K for cinder concrete is also given, Table VI, which is, however, recommended to be used only for slabs. Cinder concrete, though an excellent fireproofing material, lacks strength and should not be used as a structural material for other parts than slabs between beams.

TABLE VI.

$$r = \frac{Es}{Ec} = 35$$

p	x	K	C	S	K	C	S
.0005	.170	7.5	94	16000	7.5	94	16000
.0010	.232	14.8	138	"	14.8	138	16000
.0015	.276	21.8	174	"	18.8	150	13800
.0020	.311	28.7	206	"	20.9	"	11633
.0025	.340	33.9	225	15300	22.6	"	10200
.0030	.365	36.1	"	13688	24.0	"	9125
.0035	.387	37.9	"	12439	25.3	"	8293
.0040	.407	39.6	"	11447	26.4	"	7631
.0045	.425	41.0	"	10625	27.4	"	7083
.0050	.442	42.4	"	9945	28.3	"	6630
.0055	.457	43.6	"	9348	29.1	"	6232
.0060	.471	44.7	"	8831	29.8	"	5888
.0065	.484	45.7	"	8377	30.4	"	5585
.0070	.497	46.7	"	7988	31.1	"	5325
.0075	.508	47.5	"	7620	31.6	"	5080
.0080	.519	48.3	"	7298	32.2	"	4866
.0085	.529	49.0	"	7001	32.7	"	4668
.0090	.539	49.7	"	6738	33.2	"	4492
.0095	.548	50.4	"	6489	33.6	"	4326
.0100	.557	51.0	"	6266	34.0	"	4178
.0105	.565	51.6	"	6054	34.4	"	4036
.0110	.573	52.1	"	5860	34.8	"	3907
.0115	.581	52.7	"	5684	35.1	"	3789
.0120	.588	53.2	"	5513	35.5	"	3675
.0125	.595	53.7	"	5355	35.8	"	3570
.0130	.602	54.1	"	5210	36.1	"	3473
.0135	.608	54.5	"	5067	36.4	"	3378
.0140	.615	55 0	"	4942	36.7	"	3295
.0145	.621	55.4	"	4818	36.9	"	3212
.0150	.626	55.7	"	4695	37.1	"	3130
.0155	.632	56.1	"	4587	37.4	"	3058
.0160	.637	56.4	"	4479	37.6	"	2986
.0165	.643	56.8	"	4384	37.9	"	2923
.0170	.648	57.2	"	4288	38.1	"	2859
.0175	.652	57.4	"	4191	38.3	"	2794
.0180	.657	57.7	"	4106	38.5	"	2738
.0185	.662	58.1	"	4026	38.7	"	2684
.0190	.666	58.3	"	3943	38.9	"	2629
.0195	.671	58.6	"	3871	39.1	"	2581
.0200	.675	58.9	"	3797	39.2	"	2531

Rectangular Beams. — In determining the size of beam required for any given case, r and the limiting values of C and S are generally given, and K can be determined for any ratio, p, of concrete to steel. The value of M is determined from the conditions of loading, span and spacing, and the width and depth of beams are to be found. Formula (a) may then be put in the more convenient form.

$$d = \sqrt{\frac{M}{Kb}} \qquad (e)$$

A value for b is assumed and the equation solved for d.

Architectural or structural reasons will often limit the width or depth and several trials may have to be made.

Slabs.—For the strength of slabs the same formulæ apply. The slab may be treated as a rectangular beam of unusual width, or it may be considered as a series of beams set one alongside the other, of a width equal to the spacing of the reinforcing rods, using one rod for each beam; or it may be considered a series of beams of a unit width, using for the area of steel the area of reinforcement per unit of width.

Check Formulæ.—It may sometimes happen that it is desired to check a given or existing beam construction either as to strength or to compliance with specifications for working stresses. In that case the following formulæ will be convenient:

$$M = pSbd^2\left(1 - \frac{x}{3}\right); \qquad (f)$$

$$M = \frac{Cxbd^2}{2}\left(1 - \frac{x}{3}\right). \qquad (g)$$

If the strength of the beam for assumed working stresses is wanted, these values of S and C are inserted in formulæ (f) and (g), and the least value of M is taken. If the values of M resulting from these equations are not equal, it indicates that the full benefit of either one of the materials is not being obtained.

If the stresses in the steel or concrete due to a given loading are wanted the formulæ would be placed in the following form;

$$S = \frac{M}{pbd^2\left(1 - \frac{x}{3}\right)}; \qquad (h)$$

$$C = \frac{2\,M}{xbd^2\left(1 - \frac{x}{3}\right)}. \qquad (j)$$

These formulæ apply to rectangular beams only. M in (b) and (c) is taken as the moment of the external forces, viz; the bending moment. The value of x can be determined from Tables IV, V and VI.

In formula (b) it will be noted that the denominator of the fraction is an expression for the area of the steel multiplied by the lever arm (distance from the center of gravity of the steel to the center of compression in the concrete). Similarly in formula (c) the denominator of the fraction is an expression for

the area of the concrete in compression times the lever arm, x again being determined by (d) and M being the bending moment due to external forces.

Tee Beams.—In most instances in building construction floor slabs occur where beams or girders are used. These slabs if cast with the beams or girders add very much to the strength of beam or girder. Some authors prefer not to consider this additional strength, but to regard it as increasing the factor of safety. This position is tenable only when the slabs are cast or built independently. If made at same time, economical design requires that the slab shall be considered.

The width of slab that may be taken as part of the beam is given by the New York regulations as "ten times the width of beam or girder," that is the stem. In the Prussian regulations

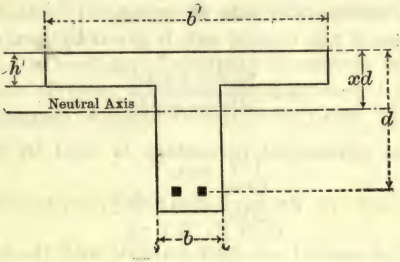


FIG. 14.

the width of flange is fixed at one-third of the span of the beam as a maximum. In any case the width of flange must not be taken as more than the distance between beams.

In an ordinary floor construction the spacing of beams, girders and columns is generally an architectural or commercial consideration. It is, therefore, generally the simplest procedure to determine at first the thickness of slab required for the given spacing of beams. This will fix the thickness of the flange of the T beam.

In the calculation of the girder, it is not objectionable to use the same slab, or as much of it as may be permissible, that has been used in the consideration of the beam framing into that girder; inasmuch as the compression stresses, in the two cases, act at right angles to and practically assist one another.

Formulæ for Tee Beams.—Fig. 14 gives a cross section of a tee beam resulting from the use of the slab as part of the

beam, and also shows clearly the notation used in the formulæ.

There may be three cases in such a construction.

Case 1.—The neutral axis may fall below the flange in which case

$$M = Spbd\left(d - \frac{h^1}{2}\right); \tag{k}$$

or

$$M = \frac{C}{2}b^1h^1\left(d - \frac{h^1}{2}\right). \tag{l}$$

In these formulæ the small area of concrete in compression below the flange has been neglected and the center of compression has been assumed at the center of the flange. This has been done to simplify the formulæ. The result is not materially affected and errs on the side of safety.

The position of the neutral axis is given by formula (m)

$$x = \frac{2bd^2pr + b^1h^{12}}{2d(bdpr + b^1h^1)}, \tag{m}$$

and the most economical percentage of steel by formula (n)

$$p = \frac{Cb^1h^1}{2Sbd} \text{ (see note).} \tag{n}$$

Case 2.—The neutral axis may coincide with the under side of the flange, in which case

$$M = Spbd\left(d - \frac{h^1}{3}\right); \tag{o}$$

and

$$M = \frac{Cb^1h^1}{2}\left(d - \frac{h^1}{3}\right). \tag{p}$$

which are the same as formulæ (k) and (l)

The economical value of p in this case is the same as in Case 1, formula (n)

Case 3.—The neutral axis may fall above the lower edge of the flange. This case is the same as Case 2, as for purposes of calculation all the concrete in the flange below the neutral axis is neglected and h becomes xd in this case as in the last.

Note.—In determining the ratio of steel to concrete in these tee beams only that part of the area of the concrete is taken that lies above the center of gravity of the steel and between the sides of the stem, that is, the area bd.

Moduli of Elasticity. — In the derivation of all these formulæ and in the determination of the values of K, the ratio of the modulus of elasticity of steel to that of concrete plays an important part. It is necessary then to know what value to use. The modulus of elasticity of steel is generally accepted as 30,000,000. The modulus of elasticity of concrete varies with many conditions. Even in the same mixture, the character of the materials affect it as well as the manner of mixing and placing. The modulus increases with the age of the concrete. It also increases with the richness of the mixture. It seems to decrease with an increase in load on the concrete.

The different values for the ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete, to be used in the design of reinforced concrete construction, as fixed by the building regulations of various cities and other authorities, is given in Table II, page 860.

Values for the modulus of elasticity of concrete under different loads and for different mixtures as determined by actual tests at the Watertown Arsenal are given in Table VII.

TABLE VII.

ELASTIC PROPERTIES OF BROKEN STONE CONCRETE 12 IN. CUBES.

Composition.			Age.	Modulus of Elasticity between loads per sq. in.			Tests Made by
Cement	Sand	Broken Stone		of 100 lbs. and 600 lbs.	of 100 lbs. and 1000 lbs.	of 1000 lbs. and 2000 lbs.	
1	2	4	7 days	2593000	2054000	1351000	*Geo. A. Kimball.
1	2	4	1 mo.	2662000	2445000	1462000	" " "
1	2	4	3 mos.	3671000	3170000	2158000	" " "
1	2	4	6 mos.	3646000	3567000	2582000	" " "
1	3	6	7 days	1869000	1530000		" " "
1	3	6	1 mo.	2438000	2135000	1219000	" " "
1	3	6	3 mos.	2976000	2656000	1805000	" " "
1	3	6	6 mos.	3608000	3503000	1868000	" " "
1	6	12	1 mo.	1376000			" " "
1	6	12	3 mos.	1642000	1364000		" " "
1	6	12	6 mos.	1820000	1522000		" " "

Working Stresses. — The working stresses for concrete and steel allowed by various cities and as recommended by other authorities, are given in Table II on p. 860. In the determination

* Tests of metals, U. S. A., 1899, p. 741.

of K the value of C, S and r as taken from Table II are substituted in formula (b), or, the value of K may be taken directly from Tables IV and V, pp. 873 to 877 and substituted in formula (e). For M in that formula the bending moment due to the external forces is used.

Bending Moments in Beams.—Beams and girders are usually considered as supported at both ends, though in many instances they are actually carried as continuous beams over the supports. If continued over the supports there is a negative bending moment produced at the support which should be taken care of by reinforcement in the upper part of the beam. This bending moment is one-half that at the middle of a supported beam, loaded at the center and two-thrids that at the middle of a uniformly loaded supported beam. In the case of supported beams loaded either at the center or uniformly, the bending moment decreases toward the support.

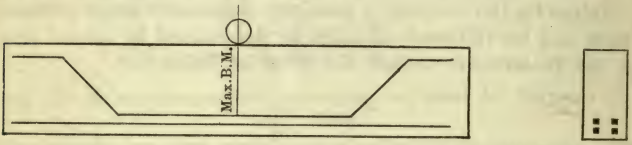


FIG. 15.

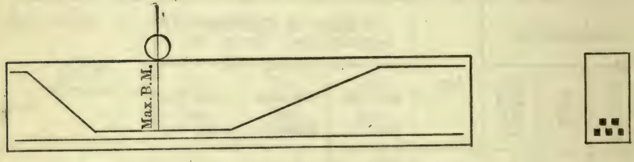


FIG. 16.

For these reasons it is well in fixing the steel to be used for the tensional reinforcement to select the bars or rods in pairs, so that as the supports are approached a part of the reinforcement may be turned up toward the top and carried across the supports near the top as indicated in Figs. 15 and 16.

Bending Moments in Slabs.—As floor slabs are usually carried continuously across the supports, the bending moment due to uniform load is assumed to be less than in beams simply supported at ends. The New York Regulations provide that "when constructed continuous and when provided with reinforce-

ment at top of plates over the supports the bending moment for uniformly distributed loads may be taken as $\frac{Wl}{10}$." The same regulations provide that for floor plates reinforced in both directions and supported on all sides, the bending moment may be assumed as $\frac{Wl}{20}$.

The following formula by Prof. C. Bach * is applicable to slabs or plates supported on four sides and reinforced in both directions:

$$M = \frac{1}{12}\,\frac{a^2 \times b^2}{\sqrt{a^2+b^2}}\,p,$$

in which M = maximum bending moment for a uniformly distributed load,
a = length of slab,
b = width of slab,
p = unit load.

Shrinkage and Temperature Stresses. — In slabs resting on or carried over two supports some reinforcement should be provided, in any case, at right angles to the tension rods to provide against shrinkage and temperature stresses. Incidentally, this reinforcement may also serve to keep the tension rods properly spaced.

Disposition of Steel. — In designing the reinforcement for any form of loading, the full sectional area required must be provided at the point of maximum bending moment. As the supports are approached, part of the reinforcement, as already indicated, is turned up, but care must be taken to keep it so distributed that at any point there is still sufficient reinforcement below the neutral axis to furnish the necessary tensional resistance. The arrangement of reinforcement for a uniformly distributed or symmetrically disposed load is shown in Fig. 15, and for a concentrated or unsymmetrical load, in Fig. 16.

In the first instance the maximum bending moment is at the center of the beam and the reinforcement is symmetrical about that point, and as much as one-half the amount of reinforcement may be turned up. In the second instance the maximum bending moment is at some other point than the center and the reinforcement must be so disposed as to get the full amount required under

* "Elasticitat u. Festigkeit" page 598. See also Concrete Engineering July 1, 1907.

the load or at the point of maximum bending moment, and the turning up must be done between that point and the support. Other conditions might require that less than half the reinforcement shall turn up. As the calculations involved in continuous beams are usually complicated, and as in most cases beams are figured as supported at ends,* the turning up of part of the reinforcement becomes largely a matter of judgment.

There is another consideration for turning up the reinforcement toward the ends, besides its value to take up negative bending moment, and that is the resistance to shear afforded by the metal running through the concrete at the points where the diagonal cracks usually occur in tests on full-sized beams.

Percentage of Reinforcement.—The amount of the reinforcement in any case is determined by formulæ (*c*) and (*n*) for rectangular and tee beams respectively. The values there obtained give the most economical amount. This may vary from one-fourth of one per cent. to one and one-half per cent. of the area of concrete, but will usually run about seven-tenths of one per cent. The nearest stock size of rods giving this amount or a slightly greater amount can be selected from the table given on page 1350, or from the catalogues of the manufacturers of the various deformed bars. The number of rods used to make up the necessary sectional area must be determined by considerations stated in the following paragraphs.

Number of Rods.—As already suggested an even number adapts itself better to a symmetrical or balanced arrangement both in cross section and horizontal section. One rod does not permit of the turning up toward the support.

Two rods may be made either to continue along the lower edge of the beam, or one may start at one support running along the lower part and turn up beyond the center as it approaches the second support and the second rod run similarly along the bottom from the second support and turn up after passing the center as it approaches the first support. Three rods may be arranged so that two continue along the bottom and the third (center one) turns up as it approaches the supports.

The arrangement for 4, 5 or 6 will naturally suggest itself from what is already said.

Too large a number of rods is not desirable as in such a case the

* The regulations of New York, Chicago, Cleveland, San Francisco, Buffalo and St. Louis, require that beams and girders shall be considered as simply supported at the ends.

rods act more or less as a screen for the coarser particles of the concrete and prevent a close contact of the concrete with the steel. This matter of complicated reinforcement is one of considerable practical importance.

On the other hand, if satisfactory encasing of the steel with concrete can be obtained, a larger number of smaller rods is preferable to a smaller number of larger rods. The area of contact in a rod of smaller size is proportionately greater than in a rod of larger size, as the perimeter varies directly as the diameter or size, and the sectional area as the square of diameter or size. In order that the rods may not slip the adhesion of steel to concrete must be equal to or greater than the tension in the steel.

Adhesion Required.—Having determined from the formulæ given the tension in a reinforcing rod at any point, we must see that in either direction from that point the steel has sufficient area of contact that the total adhesion will equal or be greater than the tension. If there is a deficiency in this respect it must be met either by mechanical bond or anchoring of the reinforcement at the ends.

Safe values for adhesion of concrete and steel are given in the table of Working Stresses on p. 860. It will be noted that all of the authorities there given, except Boston, assume the adhesion to be the same as the shearing resistance of the concrete.

A safe rule to apply, without calculation, to the case of beams with maximum bending moment at the center is to make the diameter of the rods not more than one two-hundredth of the span. Under ordinary conditions, generally speaking, the length of rod either side of the point of maximum bending moment should be at least eighty diameters for plain rods, and not less than fifty diameters for deformed bars. Under unusual conditions the adhesion should be carefully studied. The apparent discrepancy between the first and second statements of this paragraph is explained by an allowance made by the author to cover the fact that the tension in the steel does not decrease uniformly with the decrease in distance from the supports. The allowance is purely arbitrary but is considered safe. For cases of unsymmetrically loaded beams the author believes it best to examine carefully into the conditions.

Separation of Rods.—It has not been unusual in tests on beams to have the concrete split off at the under side of beams along the line of the reinforcement. This is due, if not entirely, at least in part to an insufficiency of concrete between and

around the reinforcement. To avoid this the spacing or separation of the reinforcing rods in the cross section of the beams must be such that the resistance to shear of the concrete at the level of the rods is at least equal to the adhesion of the concrete to the steel.

Assuming the safe values for shearing and adhesion to be equal, the rods should be spaced 2½ diameters on centers and two diameters from the sides of beams.

Some authorities require that the clear distance between rods and the space between rods and edges of beams shall in no case be less than one and one-half inches; * and on the other hand the Brooklyn Regulations provide that the distance need not be more than one and one-half inches.

Provision against Shear.—The low resistance to shear of concrete makes it generally necessary to provide metal stirrups to supply the deficiency. Authorities do not agree on the amount or the position of such web reinforcement. Some contend that the stirrups should be placed vertically and others contend they should be placed at an angle. The action of the stresses in the beam are analogous in the one case to those in a Howe truss where the tensile web members are vertical and in the other case to those in a Pratt truss where the tensile web members are inclined. In any case the stirrups should be long enough to reach to the top surface of the concrete.

The size and spacing of the stirrups may be determined by the following formula: †

$$a=\frac{4A}{l}\left\{(x_2-x_1)-\frac{x^2_2-x^2_1}{l}\right\},$$

in which

a = sectional area required in stirrup x_1 feet from support.

A = sectional area of horizontal reinforcement (tension rods).

1 = span in feet.

x_1 = distance from support of any stirrup.

x_2 = distance from support of next section for which area of stirrup is required.

This formula in the form given is applicable where the sectional area of any stirrup is required, the spacing of stirrups being given. When all the stirrups are to be of the same sectional area, the proper spacing may be determined by inserting the

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* Taylor and Thompson.

† See *Eng. News*, April 16, 1903, p. 348.

given value for a, assuming a value for x_1 and then solving for x_2.

Attached Shear Members.—The necessity for firmly attaching the stirrups to the tensional reinforcement is also still a matter of dispute. It is, however, safer to have them attached, as they will certainly assist in anchoring the tensional reinforcement.

Different forms of stirrups and methods of attachment are used. In the Kahn system (Fig. 8, p. 867) the stirrups form a part of the tensional reinforcement. In the Golding system the stirrup is a round rod bent in the shape of an inverted U, and secured to the tension rods by a mechanical grip (Fig. 9, p. 868). The U form, either upright or inverted, is a very common form of stirrup, and may be a round or square rod or a flat strap as shown in Figs. 7, 10 and 12.

In some cases when the slab and beams are constructed together, the slab reinforcement is carried through the upper end of the stirrups as shown in Fig. 10.

Breadth of Beam.—The breadth of a rectangular beam, and of the stem of a T beam is dependent generally on the amount of reinforcement necessary, as already indicated, and equals the sum of the diameters of the tension rods, of the required spaces between them and the amount of concrete outside of the rods needed for shear or protection to steel. In the case of beams where no stirrups are used is it also necessary to have a sufficient width of concrete to take up the horizontal shear. This should be at least equal to the sum of the perimeters of the tensional reinforcing rods.

The amount of concrete to be provided below the steel is fixed by the requirements for proper protection of the steel against fire and corrosion. (See p. 882*d*).

Compression Rods in Beams.—Steel reinforcement in the form of rods is sometimes provided above the neutral axis in beams and girders for the purpose of providing additional compressive strength where there is not sufficient concrete above the neutral axis to resist the total compression. If steel reinforcement is to be used for this purpose, the steel should be placed as high as possible, and the allowable unit compression in the steel limited to the actual compression in the concrete at that point multiplied by the ratio of the modulus of elasticity of the steel to that of the concrete, as in the case of columns with vertical reinforcement. The use of steel in compression in beams

and girders, however, is not recommended, since at best it is very uneconomical and there is a tendency for the steel to buckle and disrupt the concrete.

Columns.—Reinforced columns are of three general types: (1) Concrete with vertical reinforcement near the outer surfaces; (2) concrete wrapped with spirally wound wire or with metal bands; (3) concrete with a metal core.

Length of Columns.—The length of reinforced columns is variously limited by different authorities as follows; the figures being in each case the ratio of length to least lateral dimensions:

New York–Manhattan	12
" Brooklyn	13
St. Louis	12
Buffalo	16
San Francisco	15
Chicago	12
National Board of Fire Underwriters	12

Strength of Vertically Reinforced Columns.—In determining the strength of columns with vertical reinforcement, the steel is assumed to carry a load per square inch equal to the working load per square inch on the concrete times the ratio of the moduli of elasticity of the steel and concrete. So, for instance, in New York (Manhattan) a load of 350 pounds per square inch is allowed on the concrete, and $12 \times 350 = 4200$ pounds per square inch on the steel, 12 being the ratio of the moduli as fixed by the regulations. The allowable stresses, ratio of moduli etc. are given in Table II, p. 860.

Some authorities do not allow any load to be figured on the reinforcement, but instead allow a higher unit stress on the concrete and provide for a certain percentage of vertical reinforcement, assuming that the steel merely provides against bending.

Amount of Vertical Reinforcement.—Not less than one per cent. of vertical reinforcement should be used in reinforced columns, and it is well not to provide more than two and one-half per cent.

The reinforcing rods should be tied together horizontally at intervals of not more than the least side or diameter of the column. This prevents, to a great extent, the buckling of the

reinforcement under load and the consequent splitting of the concrete.

The vertical reinforcement to serve its purpose of taking up the bending in the column should be placed as near the outer surfaces of the column as possible, consistent with proper protection of the steel. (See p. 882.)

In the disposition of the steel the same caution is necessary as in the case of beams, to avoid getting it too close or too much of it. (See p. 881*c*.) As the concrete in columns is generally poured into the mold at the extreme top, it is particularly important to keep the interior free from interlacing steel across the column.

Brooklyn Regulations.—The regulations of the Bureau of Buildings, Borough of Brooklyn, N. Y., permit unusually high stresses on columns as follows:

"The safe load in pounds per square inch of cross section on reinforced concrete columns shall not exceed six hundred (600) for the upper story columns, six hundred and twenty-five (625) for the next lower story columns, six hundred and fifty (650) for the next lower story columns, six hundred and seventy-five (675) for the next lower story columns, seven hundred (700) for the next lower story columns, seven hundred and twenty-five (725) for the next lower story columns and seven hundred and fifty (750) for all the next lower story columns. No allowance shall be made for the steel in the columns."

Prussian Regulations.—The Prussian Regulations permit a ratio of length to least side greater than eighteen, provided that in such cases Euler's formula must be applied.

In such columns where the steel is considered as furnishing part of the compressive strength, it should be made continuous from the columns of one story into those below, or some form of washer or base should be provided at the lower end to properly distribute the load on the steel over the material below. The latter method should be avoided as much as possible, as these plates or washers tend to form planes of weakness through the concrete. The rods extending from one column may be connected with those above or below by means of pipe sleeves.

Laterally Reinforced Columns. — In the wrapped or hooped column a much higher unit strength per square inch of cross sectional area of column can be obtained, because of the effect of the wrapping. It is well known that the compressive strength per square inch of any material increases with the

area under compression or if the material is prevented from spreading laterally. The relation of this increase in compressive strength to the amount of wrapping provided has been investigated by Considere and others. In the following formula developed by Mr. F. H. Dewey of the Bureau of Buildings, Borough of Manhattan, N. Y., this relation has been taken into account.

$$P = 1.5 f_c \pi r^2 + f_h \frac{A_h}{p} \pi^2 r + 1.5 m f_c A_s,$$

in which

P = safe load in pounds,

f_c = the safe unit working stress of ordinary reinforced concrete in compression,

f_h = the safe unit tensile stress in the hoops,

r = the radius of the hoops or wrapping surrounding the concrete core,

A_h = the area of the cross section of one hoop,

p = the pitch of the hoops,

m = the ratio of the modulus of elasticity of steel to that of concrete,

A_s = the total area of the longitudinal steel.

The second part of the equation consists of three terms. The first represents the strength of the concrete itself.

The second term is the increase in strength of the concrete due to the wrapping.

The third term represents the increase in strength due to the vertical reinforcement.

The results obtained with this formula correspond very closely with the results of tests on full sized columns.

Limiting Conditions. — In figuring the strength of the columns, only the area within the limits of the wrapping can be considered as effective area. The part outside the wrapping must be treated as a protection of the steel against fire and corrosion, and must be made of the necessary thickness to secure that result. (See p. 882.)

It is to be noted that the formula is applicable only in cases where the wrapping or bonding is circular in form, and where there is sufficient reinforcement to insure a lateral resistance of at least 65 pounds per square inch. This second condition is obtained when $\frac{A_h}{p}$ is equal to or greater than $\frac{r}{250}$.

New York Formula.—When $f_c = 350$, $f_h = 16{,}000$, and $m = 12$, the formula given above becomes, approximately,

$$P(\text{in tons}) = 0.8\, r^2 + 80\frac{A_h}{p}r + 3A_s,$$

which is the shape in which it is used in New York City (Manhattan) for the working stresses there established.

Details of Lateral Reinforcement.—At the top or base of the columns in each story, the wrapping should be made to continue through the floor construction. Under certain conditions where the floor construction is practically solid about the column, affording good lateral support, equal to the wrapping, it may be better to omit the wrapping and avoid the possible complication of steel reinforcement from column, girder, and floor construction and the consequent breaking of the bond of the concrete.

The materials used for the wrapping are either steel wire or steel bands. When wire is used it is spirally wound and continuous for the full length of the column. The ends of the wire are turned into the column and turned down to such an extent that when the concrete has been poured and set, there will be sufficient anchorage to resist the tension in the wrapping due to the outward pressure of the concrete.

When metal bands are used, as in the Cummings system, care must be taken to make the rivetted joints in the bands as strong as the bands themselves.

A form of wrapping that has the merit of rapidity and ease of erection is shown in the columns used in the Bush Terminal Warehouse, Brooklyn, New York, described on p. 882*a*, in connection with fireproofing.

Metal Core Columns.—The object of this type of column is to provide a construction for tall or heavily loaded buildings that will have the necessary strength and yet not encroach too seriously on the floor space. For this form of column some engineers advocate placing a steel core in the center of the concrete, the steel taking the bulk of, if not the entire, load. *

For the McGraw Building, in New York City, a column was designed by Prof. William H. Burr, as shown in Fig. 17. The steel core is of sufficient strength as a column, independent of any concrete, to carry the entire dead load coming upon it, the stresses

* Trans. Am. Soc. C. E., Vol. XIV, Part E, p. 556.

in the steel being in no case greater than those allowed on steel columns under the New York Building Code, considering the ratio of length to radius of gyration. Furthermore, those stresses were not allowed to exceed 9,000 pounds per square inch in any

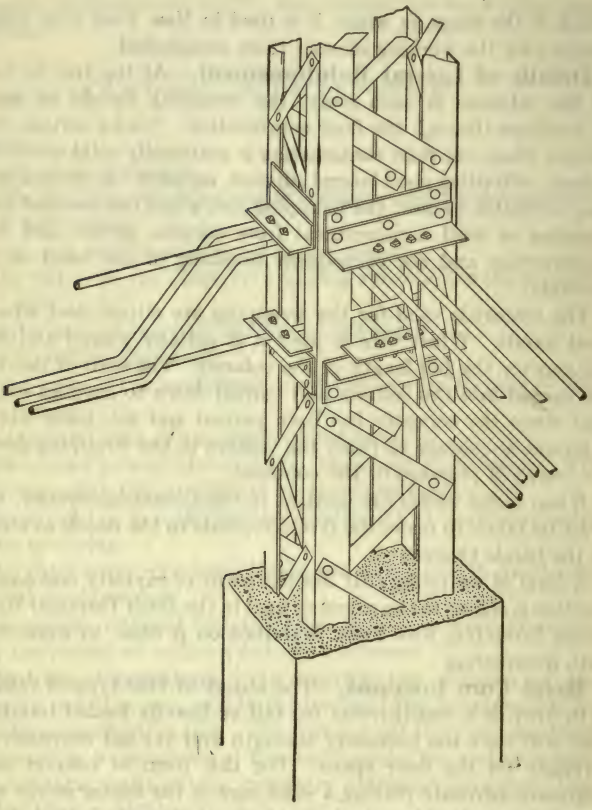

FIG. 17.—Column, and Girder and Column Connection, McGraw Building, N. Y.

case. The live loads were provided for by placing a sufficient amount of concrete within the steel framework, that the stress on the concrete did not exceed 750 pounds per square inch, being one-twelfth of the maximum allowable load on the steel. The

concrete outside of the steel was considered only as protection against fire and corrosion.

Cast-iron Columns.—Where the building for any reason need not be treated as a fireproof structure, space and time may be saved by using cast iron or steel columns. In such cases the column connections must be designed to get suitable bearing for the concrete construction and a continuity of that construction; for the great advantage in reinforced concrete construction lies in its monolithic character. When cast iron columns are used, the

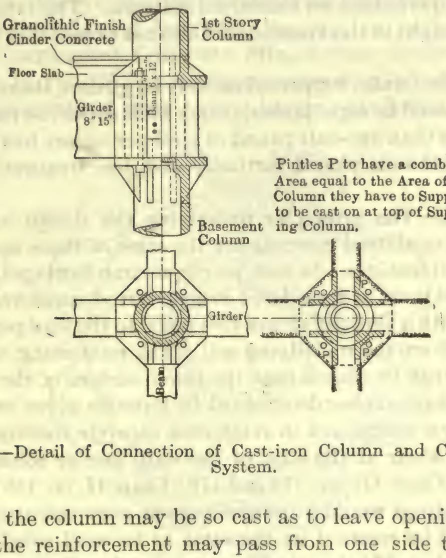

FIG. 18.—Detail of Connection of Cast-iron Column and Concrete Floor System.

head of the column may be so cast as to leave openings through which the reinforcement may pass from one side to the other Fig. 18 shows how this has been done, in a building at Gay and Christopher Streets, New York, without impairing the strength of the column at the connection.

Steel Columns.—In steel columns it would be simpler to provide a connection of the reinforcing rods with the steel shapes of the columns. Where the reinforcement does not go through the columns, some rods should be placed outside the columns to tie as much as possible the concrete on one side to that on the other.

Eccentric Loads.—When the loads on columns are eccentrically placed additional sectional area must be provided than is

required for direct load. A formula for eccentric loading is given in Chap. XIV on p. 413.

Walls.—Concrete walls are generally considered better than brick or stone walls. If not reinforced they are generally required to be of the same thicknesses, for given conditions, as brick walls. Under such circumstances they are probably not as economical as brick walls.

If reinforced and used as bearing walls, they can be reduced to about two-thirds the thickness of brick walls, provided, however, that the load on the concrete does not exceed the safe load per square inch permitted on reinforced columns. The ratio of unsupported height to thickness should also not exceed that fixed for columns.

For spandrel walls, supported entirely on girders, the minimum thickness should be eight inches. Such walls should be reinforced with not less than one-half pound of steel per square foot of wall, in the form of rods placed vertically and, less frequently, horizontally.

Footings.—The principles underlying the design and construction of reinforced concrete are the same as those applied to other types of footings. In wall, pier or column footings the overhang or offset must be considered as an inverted cantilever loaded uniformly with a load per square foot equal to the load per square foot imposed on the underlying soil. The reinforcing rods will then necessarily be placed near the lower surface of the footing and the size and number determined by formulæ given on p. 872. A detail often overlooked in reinforced concrete footings is the tendency to shear at the edge of the wall, pier or column supported. (See Chap. III, pp. 178 and 179; Chap. II, pp. 158 to 161.)

Where footings would otherwise become very eccentric cantilevers should be resorted to, the same as for steel construction. (See Chap. II, p. 174 *et seq.*) The maximum bending moment on the cantilever is determined and the concrete girders designed as described on p. 872.

Economy of Reinforced Concrete Footings.—Great economy over steel grillage or other types of footings may often be effected in the use of reinforced concrete footings. The cost of the latter type will vary from twenty to forty per cent. of the cost of a corresponding steel grillage footing. This difference is easily accounted for. The amount of excavation for the reinforced footing is generally much less than for the steel grillage. A smaller amount of concrete is used, and this concrete is con-

sidered in the calculations for strength, whereas in the steel grillage, the concrete is chiefly provided for encasing and protecting the steel. The amount of steel is much less, being used only to supply the tensional resistance of the construction; the compressive strength being supplied by the much cheaper material, concrete. Incidently, the protection of the steel in the reinforced footing is generally more certain than in the steel grillage footing.

Concrete Piles.—Concrete piles have been referred to in Chap. II, p. 177 and the principal commercially successful types have been those mentioned. Other types have been developed, of which the following deserve notice.

The Corrugated Concrete Pile, a cross section of which is shown in Fig. 19, is a reinforced concrete pile, cast in forms on the ground and after having hardened sufficiently is driven

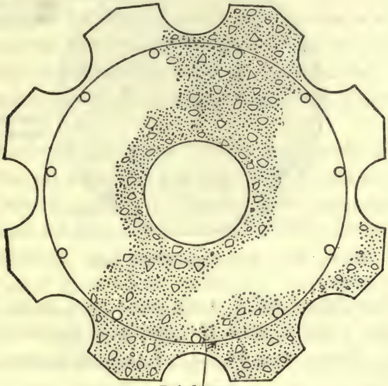

FIG. 19.—Corrugated Pile.

by means of a water jet working through the central core- These piles are made by the Corrugated Concrete Pile Company, New York.

The Chenoweth Concrete Pile, a cross section of which is shown in Fig. 20, is constructed without forms by rolling up around a central mandril a sheet of expanded metal or wire netting upon which has been spread a layer of concrete. Longitudinal reinforcing rods are tied to the expanded metal or wire netting at intervals as shown in the cut. This pile may be driven

by either hammer or water jet. Mr. A. C. Chenoweth, 16 Court Street, Brooklyn, New York, is the patentee. *

Loads.—No accepted formula for the strength of piles has as yet been offered. The formula used for wooden piles (p. 154) is not applicable. In lieu of anything more satisfactory, the permissible loads on concrete piles may be determined in the same manner as the loads on concrete columns, p. 881*g*. Concrete piles should be driven to a solid bearing, and should not depend on frictional resistance. A few test loads on concrete piles are here given.

At Annapolis, Maryland, a Raymond concrete pile 6″ at the point and about 18″ at the butt, 17½ feet long, was loaded with 41 tons with a total settlement at the end of one month of 3–32″. Another Raymond pile at the same work, driven in reclaimed

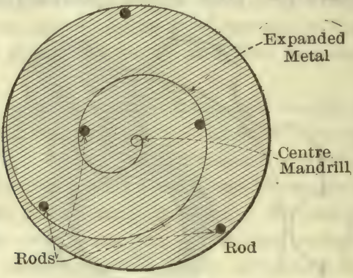

FIG. 20.—Chenoweth Pile.

land which had been filled with sand and mud three years previously, was loaded with 42 tons, and showed a settlement of 0.002 feet. There was no additional settlement at the end of one month.†

For the foundation of the Produce Exchange Building, Broadway and Beaver Street, New York, a Simplex concrete pile was loaded with about 43 tons ‡ with a settlement of 3–16″ at the end of six days.

Connections.—Much judgment can be displayed and must be exercised in the design of details in the connections. The great value of reinforced concrete construction over other types is the possibility of securing great rigidity. This can only be attained

* See *Eng. News*, July 26, 1906, p. 105.

† *Eng. Record*, March 4, 1905, p. 277.

‡ Under the auspices of the Bureau of Buildings.

when the result is as nearly monolithic as possible. We then have mass to take up vibration and this advantage in the case of workshops or factories employing moving machinery is readily seen.

The reinforced concrete buildings that came through the severe San Francisco earthquake in May, 1906 in good condition, it has been found, were those in which attention had been given to the details and connections.

To secure monolithic character requires not only continuity in the concrete, but a continuity in the reinforcement. This often means that a net work of steel occurs at the connections. If carried to excess, the bond and continuity of the concrete is apt to be broken, even when the spaces between steel are thoroughly filled. But when there is such a net work of steel it also acts like a sieve and the spaces are not readily filled. For this reason it is well to use a richer mixture at the columns and to keep the aggregate as small as possible.

The connections of floor system to columns are particularly troublesome in this respect, and partly for this reason, as well as to provide rigidity, brackets should be provided under the girders at the columns with metal reinforcement near the inclined surface of the bracket.

Stairs.—Some of the most interesting work that has been done in reinforced concrete has been the construction of stairs. The reinforcement, being in the form of comparatively small limber bars, can be adapted to most any shape for which molds can be constructed, and when a wet, rich concrete with small aggregate is used little or no difficulty need be experienced in casting. As an example of such work, the stairs in the residence of Mr. G. W. Vanderbilt, in New York City, may be cited. When this stair was five weeks old a test of its strength was made by dropping a bundle of four bags of cement (about 380 pounds) from the floor above to the intermediate platform, a distance of eleven feet. Not the slightest disturbance occurred. *

Types of Construction.

Mill Construction. — In localities where labor is high and where conditions are more or less congested, it is probably more economical to use brick for walls than concrete. In such

* For a detailed description see *Cement* for Jan., 1904 and *Eng. Record*, Dec. 12, 1903. For other examples of stair-work see *Eng. News*, June 30, 1904.

cases the type of construction is similar to ordinary mill construction. Provision must be made to anchor the beams and girders which can be done by bending the ends of the reinforcing rods so as to extend horizontally into the wall each side.

Skeleton Construction. — The skeleton type of construction seems to be the best form adapted to reinforced concrete. A framework of columns, girders, beams and flooring is built, as in steel construction, the wall girders and columns, of course, being designed to carry the weight of the outside walls as well as such portion of the floor and live loads as come on them. The work, in this type of construction, can progress more steadily than in the mill construction, as a rule, since the concrete work need not at any time be stopped to wait for the brick work to be carried up, if brick is used for the walls.

In the skeleton construction any type of outside wall may be used; brick, concrete, tile, etc. In some cases the panels are simply filled in with brick work, eight or twelve inches thick, leaving the concrete columns and girders showing between the brick panels. In walls situated on property lines, where there is a likelihood of adjoining buildings this is not objectionable. If the wall remains exposed and appearance is a consideration, the columns and girders can be treated architecturally to set off the brick work. Or the brick work may be continued as a facing over the outside of the columns and girders as was done in the Bush Terminal Warehouses in Brooklyn, New York. * To thoroughly secure this brick facing, galvanized anchors were placed in the concrete columns and girders as they were being erected, projecting sufficiently to bond into the brick joints.

In using concrete for the panels, the walls, properly reinforced, may be six and, in cases, even four inches thick. The sides of the columns are cast with pockets, grooves or recesses to receive the panels, which as in the case of brick work, are most satisfactorily and most economically built after the removal of the molds from the skeleton frame.

In the Marlborough-Blenheim Hotel, at Atlantic City, New Jersey, the panels are filled in with hard burned terra cotta tile and a stucco applied on the outside. This makes a comparatively light construction and affords good insulation.

The particular advantage in the skeleton type of construction,

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* For description of this building, see *Eng. Record*, March, 3, 1906.

especially in workshops or factories, is the possibility of large window areas affording light and ventilation.

System "M."—A type of construction known as their "System M" has been developed by the Standard Concrete Company of New York. (Fig. 21.)

It consists of a light steel skeleton designed to carry the dead load of the entire structure (except that the columns are designed for gross loads) which is then encased in concrete making ultimately a reinforced concrete construction. *

Its advantage consists in the ability to erect and inspect the steel reinforcement before even the centers or molds are placed. Under congested conditions, such as prevail in large cities, particularly New York, it is a rapid form of construction. The use of the steel in this type is, however, not economical. In order to get the necessary strength in the steel framework, shapes must be used which do not offer the amount of adhesion that should be had for the amount of metal. Furthermore such shapes must necessarily be subjected to some bending which would tend to break the bond between concrete and steel.

Mushroom System. — In the so-called "Mushroom" system, invented and patented by Mr. C. A. P. Turner of Minneapolis, Minn., an attempt is made to eliminate all beams and girders and to construct floor slabs supported directly by the columns and walls. The columns at the top are spread out into a cap, reinforced circumferentially and radially. The principal slab reinforcement runs diagonally across the slab from column to column, in two directions. A secondary reinforcement and somewhat lighter reinforcement runs from column to column along the outer lines of each floor panel.

It will be seen that this system allows the use of a shallower floor construction than is ordinarily attainable. In erection, too, it has the advantage of a flat centering for the entire floor.

Kahn Hollow Tile Construction.—In seeking to minimize the cost of centering, the floor construction shown in Fig. 22 has been devised. It consists of a series of reinforced concrete beams with a clear space between them of the width of hollow tile blocks. In erection, a flat centering may be used, which, however, need not even be continuous. Planks, a few inches wider than the concrete beams, are placed under the

* For a full description see *Eng. News*, April 25, 1907, and *Eng. Record* June 22, 1907.

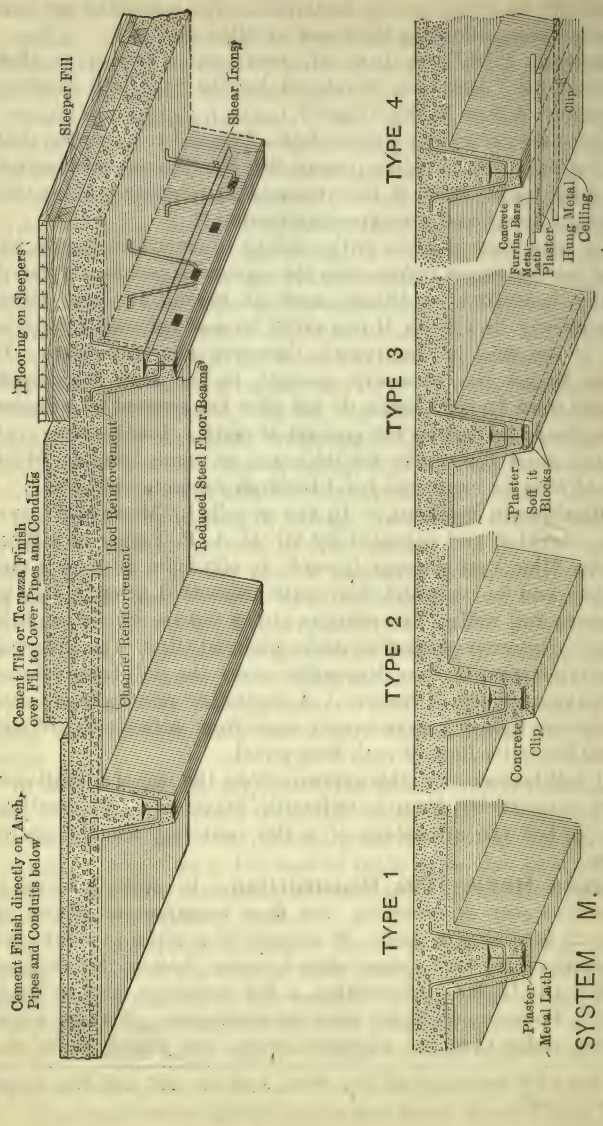

Fig. 21.—System "M."

location of the beams, the tiles are laid in rows supported along their edges by the planks and thus form the sides of the molds for the beams. The reinforcement is placed and the concrete poured with or without a floor plate, as the necessities of the case may require. Care must be taken in pouring the concrete that the tiles are not displaced sidewise. The tile should also fit closely at their joints, otherwise the finer particles of the concrete are liable to flow into the tile, either making a porous concrete or using more cement and sand than necessary.

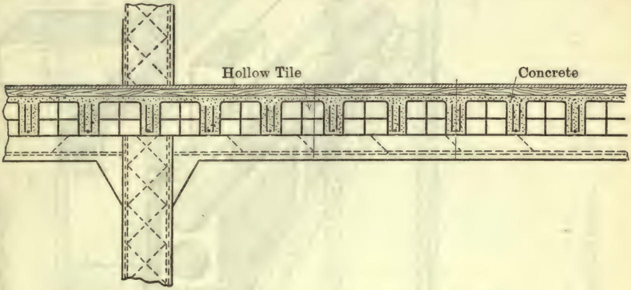

Fig. 22.—Hollow Tile and Concrete Floor.

This form of construction offers besides the economy in centering, the advantages of a flat ceiling without the application of lath and, in roof construction particularly, the avoidance of condensation.

Faber Construction.—The same principle of construction is involved in the Faber Construction (patent held by Faber Construction Company, New York), recently introduced in the country from abroad where it has been used extensively. (See Fig. 23.) In this case, however, the floor is reinforced in both directions, and the strength calculated as a slab supported on four sides. (See p. 881*b*.) The concrete is a rich mixture of one part Portland cement and three parts sand. It is prevented from running into the hollow spaces of the tile by the use of the cardboard tubes as shown. The tiles are of heavier construction than the ordinary commercial tile, and are used in part in figuring the resistance to compression.

Merrick System.—Another type of construction, known as the Merrick System, controlled by Ernest Merrick, 11 Broadway, New York, is shown in Fig. 24. *

* See also chap. XXIII, p. 795.

It consists of a series of reinforced concrete beams connected by a concrete plate at the top and a ceiling plate at the bottom.

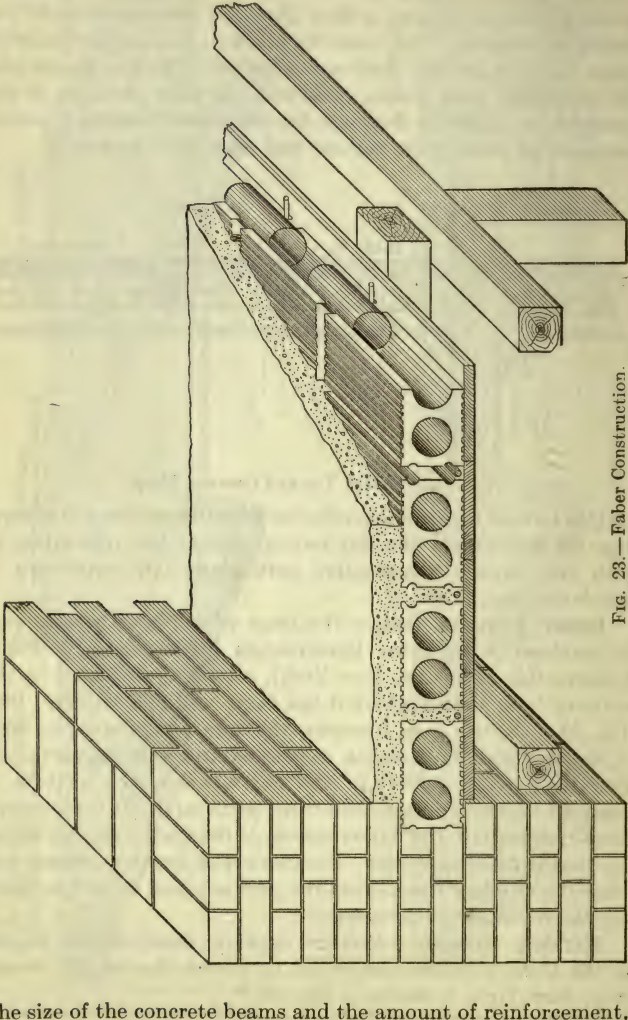

Fig. 23.—Faber Construction.

The size of the concrete beams and the amount of reinforcement, are dependent on the span and loading. The special features of

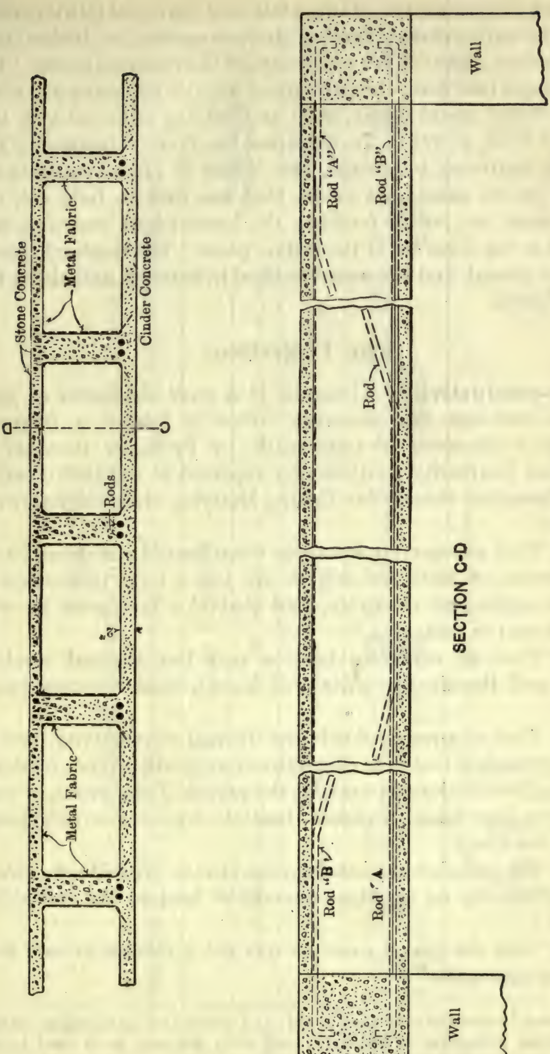

Fig. 24.—Merrick System.

the construction are the ceiling plate and the metal fabric centering. The ceiling plate is made of cinder concrete, two inches thick and offers an effective fire protection to the concrete beams. The metal cages that form the permanent interior centering are made of a stiffened metal fabric, such as Roebling stiffened lath (see Chap. XXIII, p. 827). To construct the floor, a temporary flat wooden centering is erected, two inches of cinder concrete are placed on the same, and before that has time to fully set, the metal cages are put in position, the lower edges becoming embedded in the concrete of the ceiling plate. The reinforcing rods are then placed, and the concrete filled in between and above the metal fabric.

Fire Protection.

Non-conductivity.—Concrete is a poor conductor of heat and in this fact lies whatever virtue it has as a fireproof material.* A series of tests made by Professor Woolson of Columbia University, and recently reported at the 1907 meeting of the American Society for Testing Materials show the following results:

(1) "That all concrete mixtures when heated throughout to a temperature of 1000° to 1500° F. will lose a large proportion of their strength and elasticity, and that this fact must be well remembered in designing."

(2) "That all concretes have a very low thermal conductivity, and therein lies their well known heat resisting properties."

(3) "That as a result of this low thermal conductivity, two to two and one-half inches of concrete covering will protect reinforcing metal from injurious heat for the period of any ordinary conflagration (provided, of course, that the concrete stays in place during the fire).

(4) "That reinforcing metal exposed to the fire will not convey by conductivity an injurious amount of heat to the embedded portion."

(5) "That the gravel concrete was not a reliable or safe fire-resisting aggregate." †

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* It must be remembered that in this and succeeding paragraphs on the fire-resisting properties of concrete, only such material as is used in reinforced concrete is considered. The value of cinder concrete as a fireproof material is considered in the previous chapter, p 734.

† *Eng. News*, Aug. 15, 1907, p. 168.

TABLE VIII.

CONCRETE BLOCKS HEATED ON ALL SIDES.*

Specimens 6″×6″×14″ Prisms—Proportions 1:2:4 Age 2 Months—Temperature 1500° F.

Aggregate.	Modulus of Elasticity.								Breaking Load in Pounds per sq. inch.		
	At 200 lbs. per sq. in.			At 400 lbs. per sq. in.			At 800 lbs. per sq. in.				
	Unheated.	Heated. 3 hrs.	Heated 5 hrs.	Unheated.	Heated. 2 hrs.	Heated. 5 hrs.	Unheated.	Heated. 3 hrs.	Unheated.	Heated. 3 hrs.	Heated. 5 hrs.
Limestone..	6,000,000	200,000	....	6,000,000	285,000	....	5,647,000	425,000	2,740	1,345	870
Trap rock..	3,430,000	150,000	129,000	4,355,000	222,000	188,000	4,355,000	348,000	3,140	1,400	997
Cinder.....	1,090,000	49,500	571,000	960,000	....	....	915,000	....	1,400	547	504
Gravel.....	8,000,000	....	....	6,887,000	....	....	6,000,000	....	2,780	....	...

* Proc. Am. Soc. Testing Materials, Vol. VI, p. 446.

Loss of Strength.—If its non-conductivity were all that is involved in the fireproof character of concrete, the minimum thickness required for the protection of the steel would be easily determined. But the strength of the concrete is more or less affected when exposed to the extreme heat of fire. An effort has been made to determine this effect and a summary of the results as reported by Professor Woolson of Columbia University * is given in Tables VIII and IX

TABLE IX.

CONCRETE BLOCKS HEATED ON ONE FACE ONLY. †

Specimens 6″×6″×14″ Prisms—Proportions 1:2:4.

Age 2 Months—Temperature 1500° F.

Aggregate.	Modulus of Rupture after heating 5 hours.			Breaking Load in lbs per sq. in.
	At 200 lbs. per sq. in.	At 400 lbs. per sq. in.	At 800 lbs. per sq. in.	
Limestone.........	293400	521700	730700	1840
Trap rock.........	200000	268000	379000	1705

Fire Tests.—The effect of fire on reinforced concrete has also been studied in a number of tests made by the building authorities of New York City and Philadelphia, and in some of the recent conflagrations in the country, notably at San Francisco.

The test to which the sample full size constructions have been subjected, is described in the previous chapter, p. 745, except that there was no limitation as to span or spacing. Of the nineteen tests made under the auspices stated, seven were failures, the others passing in more or less satisfactory condition.‡

The conclusion, from a study of the tests in detail,§ shows that to a depth averaging about one inch the concrete is seriously impaired and is easily washed off by a hose stream applied to the

* Proc. Am. Soc. Testing Materials, Vol. VI, p. 433.

† Proc. Am. Soc. Testing Materials, Vol VI, p 448.

‡ For a partial lise of these tests see Table in Proc. Am, Soc. T. M., Vol. VI, p. 128. Several tests have been made since that report was submitted.

§ The detailed reports are on file in the Bureau of Buildings, Borough of Manhattan, N. Y.

surface. Any stone containing an appreciable percentage of carbonate of lime will calcine and cause failure. Where the construction is poorly designed, allowing an excessive deflection, the fine cracks in the concrete below the steel will open to such an extent as to allow the heat to reach the metal reinforcements When the reinforcement is such as to produce a plane of weakness in the concrete there is liable to be a flaking off of concrete and a consequent exposure of metal.

Actual Fire.—The earliest test of a reinforced concrete building in an actual fire occurred in 1902, in the four story factory of the Pacific Coast Borax Company, at Bayonne, New Jersey. The roof of this building was of wood, which, with the contents of the building, was destroyed by the fire. The only damage suffered was a break in the top floor caused by the fall of a heavy tank that had been supported by the roof. At the same time an adjoining building of unprotected steel posts and beams was twisted into a tangled mass of metal.

Baltimore Fire.—In the Baltimore fire there was but one reinforced concrete building of the three exposed to the fire from which any fair conclusion can be drawn. In one of the buildings, the concrete construction was entirely destroyed, but this was probably due to the falling walls and failure of other non-fireproof parts. In a second building the heavy floor of a banking room of reinforced concrete came out practically unharmed, but it was not exposed to severe fire. The third structure was, however, exposed to severe fire. The contents of the building were destroyed, and a large part of the outside walls fell. The floors, five in number, were all of reinforced concrete supported on concrete columns, having replaced an old joisted construction. A test made after the fire showed that the floors were still capable of sustaining the loads for which they were designed, although the floor slabs were cracked. The girders were cracked longitudinally near the line of the reinforcement, and the columns were spalled to such an extent as to expose most of the reinforcement. It would have been difficult to restore the building so that it would resist another such attack. *

San Francisco Fire.—The effects of the fire on concrete construction in the conflagration immediately following the San Francisco earthquake in 1906, are summed up in the following

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* Capt. Sewell in his report on this building draws a different conclusion. See *Eng. News*, March, 24, 1904, p. 276.

paragraph from the report of a committee of engineers investigating the subject.

"Concrete floors generally had hung ceilings, and, where thus protected, were uninjured. Where exposed, the concrete is in most cases destroyed, for instance, in the Sloan, Rialto, and the Aronson Buildings, and the Crocker Warehouse. The concrete is dry, and while in many cases hard, yet all the water has been burned out and it may be said to be destroyed, even if able to support weights. Floor coverings of wood invariably burned, adding to the destruction. Sleepers were generally burned. Surfaces of cement mortar fared much better, the linoleum covering remaining practically intact." *

In discussing the report, Mr. A. L. A. Himmelwright, who made a personal inspection of the ruins, concludes that reinforced concrete is inferior as a fire-resisting construction to any form of steel construction with concrete floors and concrete column and girder protection, but superior to steel construction with terra cotta floor and terra cotta column and girder protection. "Where this method was used, a very slight attack of fire was generally sufficient to cause the rupture of the concrete underneath the reinforcing metal, so that it fell away, exposing the metal. There were comparatively few buildings, however, in which this method of construction was used." †

Thickness of Concrete Required.—From a study of the tests and fires just referred to, a fair conclusion as to the amount of protection against fire would seem to be: In all columns, in large and important girders, trusses or other supports at least two inches of concrete outside of all reinforcement; in girders and beams and slabs of long spans, about one and one-half inches of concrete outside of all reinforcement; in stair work, floor slabs (short span), walls and partitions from three-quarter to one inch of concrete outside of all reinforcement.

In footings and foundations the thickness of concrete outside of reinforcement should be at least three inches. Not for fire protection, but for protection against corrosion. (See p. 882*d*.)

Fire Underwriters' Requirements.—The following provisions are those given in the Building Code recommended by the National Board of Fire Underwriters. They are generally considered, except by underwriters and some fire engineers, as ultra conservative.

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* Proc. Am. Soc. C. E., March, 1907, p. 330.

† Proc. Am. Soc. C. E., August, 1907, p. 668.

"The minimum thickness of concrete surrounding and reinforcing members one-quarter inch or less in diameter shall be one inch; and for members heavier than one-quarter inch the minimum thickness of protecting concrete shall be four diameters taking that diameter, in the event of bars of other than circular cross-section, which lies in the direction in which the thickness of the concrete is measured; but no protecting concrete need be more than four inches thick for bars of any size; and provided, further, that all columns and girders of reinforced concrete shall have at least one inch of material on all exposed surfaces over and above that required for structural purposes; and all beams and floor slabs shall have at least three-quarters inch of such surplus material for fire-resisting purposes."

Other Forms of Protection.—Because of the effects produced by fire on reinforced concrete, as above described, and the difficulty of restoring the construction where so affected, various suggestions have been made to protect the concrete construction with other materials.

On account of its excellent fire-resisting qualities (see p. 734), cinder concrete would naturally suggest itself. This material is out of the question where strength is required. But its use may be combined with that of stone concrete, by placing a sufficient thickness for protective purposes, in the outside of the reinforcement in columns, below the neutral axis in beams and girders, * and on the under surface of floor slabs.

Difficulties are likely to be met in placing two kinds of concrete in the same mold, but they are not insurmountable. But careful inspection is required to see that the poorer material is not put in place of the stronger. One kind of concrete should also follow the other immediately in order to secure a bond between the two.

The application of this suggestion to column protection was satisfactorily applied in the Bush Terminal Warehouses in Brooklyn, serving at the same time another purpose. (See p. 881*j*.)

The steel wire wrapping for the columns was prepared in sections of two feet height. A metal lath about one-half inch mesh was placed outside the wrapping and secured to it. This was then placed in a cylindrical wooden mold two feet high and of a diameter four inches larger than the wrapping, and

* Trans. Am. Soc. C. E. Vol. LVI, p. 284.

formed the inner side of the mold. The space between the wrapping and the wooden mold was then filled with cinder concrete. When set and the mold removed, the result was a hollow cylinder of cinder concrete, two inches thick and two feet height, with the column wrapping attached to the inside. These cylinders were set one over the other in the building till the proper column height was reached, such vertical rods as were wanted were put in and the interior filled with concrete. Thus was produced a fireproof wrapped column without the expense and inconvenience of column molds in the building. (Fig. 25.)

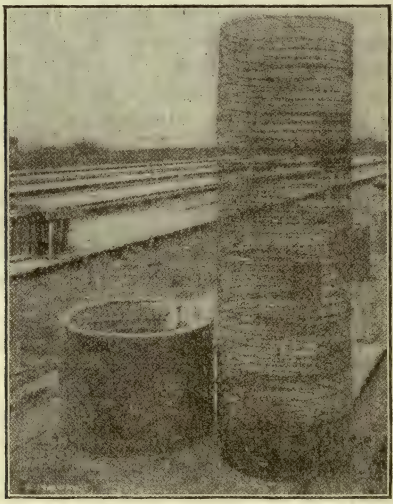

FIG. 25.—Fire-proofing Shell for Columns.

In the "Merrick System" (see p. 881*t*) a protection to the lower side of a floor construction is sought, giving at the same time what is often desirable, a flat ceiling.

A form of fire protection, advocated by the National Fire Proofing Company of Chicago, is shown in Fig. 26. Here columns, beams and girders are completely encased with hollow tile blocks. Being either laid in the molds or forming them, their rough and furrowed porous surfaces cause them to adhere firmly to the concrete. This affords as great protection as for steel columns, and if destroyed the blocks can be replaced

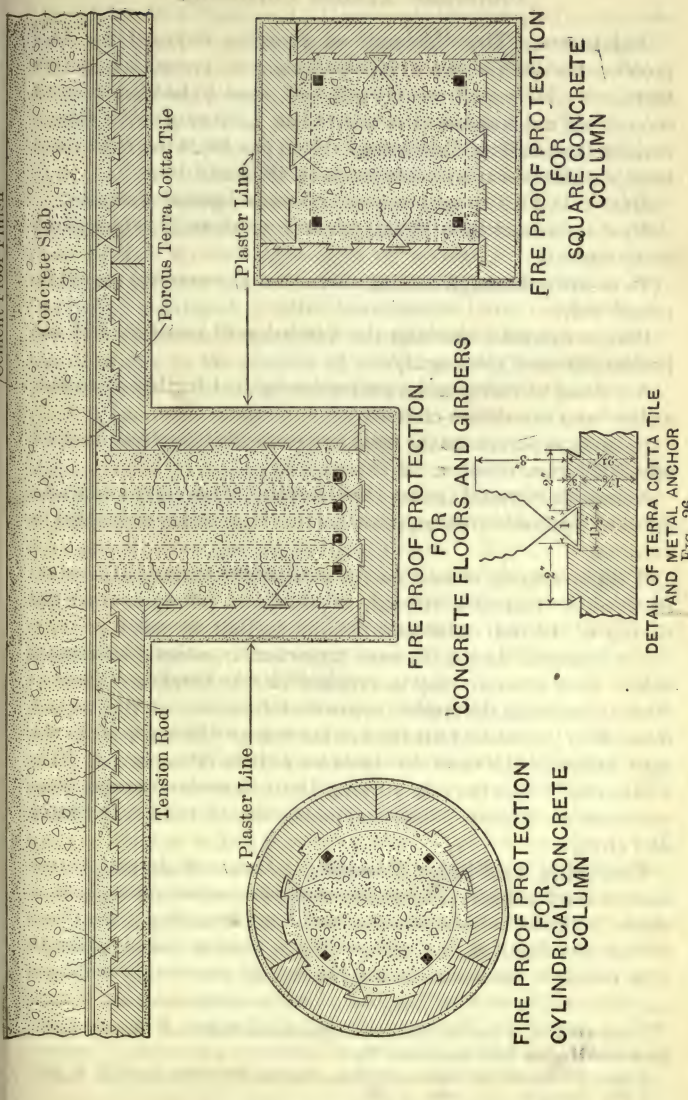
Cement Floor Finish
Concrete Slab
Porous Terra Cotta Tile
Plaster Line
Tension Rod
Plaster Line
FIRE PROOF PROTECTION FOR SQUARE CONCRETE COLUMN
FIRE PROOF PROTECTION FOR CONCRETE FLOORS AND GIRDERS
FIRE PROOF PROTECTION FOR CYLINDRICAL CONCRETE COLUMN
DETAIL OF TERRA COTTA TILE AND METAL ANCHOR

FIG. 26.

Protection Against Corrosion.

Thickness.—The thickness of concrete required for fireproofing has been found to be also ample for protection against corrosion. It is well established that steel imbedded in neat cement will not corrode. Professor Chas. L. Norton of the Massachusetts Institute of Technology draws the following conclusion from a series of experiments made in 1902 and 1903. *

Steel imbedded in neat cement is secure against corrosion.

Steel imbedded in a dense concrete mixture is safe against corrosion.

To assure a thorough coating of the steel the concrete should be mixed wet.

Porous concrete, allowing the admission of moisture will not protect the steel thoroughly.

A coating of rust is not a protection against further corrosion, as has been sometimes claimed.

In these experiments the steel was encased in concrete one and one-half inches thick on all sides.

From this it would appear that the steel of reinforced concrete is secure against corrosion, provided it is thoroughly imbedded in concrete.

A slight coating of rust on the steel where imbedded does no harm as the cement is strongly alkaline and will counteract the acidity of the iron oxide and prevent further corrosion.

"In practical design the most important question which arises is how far a concrete may be cracked (due to bending of beams) without exposing the steel to corrosive influences. In this respect it seems to the writer that the minute cracks which appear in the early states of the tests can have very little influence." † This means that within the safe working limits, there is no danger from corrosion on account of the fine cracks due to tension in beams and girders.

Corrosion in Cinder Concrete.—Cases of serious corrosion of steel imbedded in cinder concrete are on record. In a report to the Structural Association of San Francisco ‡ the Committee investigating the subject states that in cinder concrete "the extent of the corrosion is great enough to seriously endanger

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* Reports Nos. 4 and 9, Insurance Experiment Station of Boston Manufacturers Mutual Fire Insurance Co.

† Prof. Turneaure in Trans. Am. Soc. Testing Materials, Vol. IV, p. 505.

‡ *Eng. News*, Nov. 1, 1906, p. 458.

the safety of the floors, and it is not probable that the floors would have supported their loads more than one to three years longer." The Committee recommended "that the Structural Association try to amend the present building law so as to exclude the use of cinder concrete in floor slabs or for fireproofing."

Mr. William H. Fox in his investigations * on this same subject finds that "after about forty days' treatment, the specimens were broken, and the steel carefully examined for corrosion. With but one exception, one or more of the three steel pieces in each specimen showed unmistakable signs of corrosion. Apparently it made no difference how the concrete was mixed—wet or dry, tamped or untamped, whether the steam or water treatment was used, the result was the same—rust streaks and spots were found; the difference in the amount of corrosion being imperceptible." He concludes that "to secure a dense homogeneous cinder concrete, a thorough tamping is necessary. A rich mixture, either 1:1:3 or one in which the proportion of cement to aggregate is larger, should be used in all cases. The greatest of care should be taken in mixing the materials, and it may be necessary to resort to the seemingly impractical method of coating the reinforcement with grout before placing in the concrete."

On the question of the corrosion of steel in cinder concrete Prof. Norton concludes: "There is one limitation to the whole question, that is, the possibility of getting the steel properly incased in concrete. Many engineers will have nothing to do with concrete because of the difficulty in getting 'sound' work. This is especially true of cinder concrete, where the porous nature of the cinders has led to much dry concrete and many voids, and much corrosion. I feel that nothing in this whole subject has been more misunderstood than the action of cinder concrete. We usually hear that it contains much sulphur and this causes corrosion. Sulphur might, if present, were it not for the presence of the strongly alkaline cement; but with that present the corrosion of steel by the sulphur of cinders in a sound Portland concrete is the veriest myth, and as a matter of fact the ordinary cinders, classed as steam cinders, contain only a very small amount of sulphur. There can be no question that cinder concrete has rusted great quantities of steel, but not because of its sulphur, but because it was mixed too dry, through the action of the cinders in absorbing moisture, and that it contained, therefore, voids; and secondly, because in addition the cinders often contain

* *Eng. News*, May 23, 1907, p. 569.

oxide of iron which, when not coated over with the cement by thorough wet mixing, causes the rusting of any steel which it touches. There is one cure and only one, mix wet and mix well. With this precaution I would trust cinder concrete quite as quickly as stone concrete in the matter of corrosion." *

In 1902 the Pabst Building in New York City, an eight-story steel skeleton construction, was taken down after having been erected about four years. The floor filling between the steel I beams in this case consisted of cinder concrete on metal lath, in segmental form. (Roebling construction.) The steel work generally was found to be free from rust, though it should be remembered that all the steel had been painted.†

Taking all things into consideration it would probably be safe to use cinder concrete, if care is taken to provide a proper mixture and careful and thorough workmanship.

Erection.

Forms.— For the erection of reinforced concrete, it is generally necessary first to construct molds or centerings for the columns, floors, etc. Wood is the material used for the purpose. Sheet metal centering has been used with questionable success and economy.

In the selection of the wood for the molds a clean grade of dressed pine should be sought. It should be thick enough to resist warping and to resist deflection between supports. It must be coated on its surface with soap or some other satisfactory substance to prevent it from sticking to the concrete.

The forms or molds must be erected carefully to exact size of proposed parts and true to position and direction. For floor molds sufficient supports must be provided not only to carry safely the heavy wet concrete, but also such materials as are liable to be placed on the floors up to the time when the concrete has set sufficiently to carry such loads. The supports must have sufficient rigidity to prevent deflection in the molds.

The molds should be so constructed that they can be easily removed when the concrete has set.

Sharp corners should be avoided as much as possible, as the wood is liable to stick in them. Where re-entrant angles in the

* Report No. 9 Insurance Experiment Station, Boston Manfrs. Mut. Fire Ins. Co.

† Trans. Am. Soc. C. E., Vol. L, p. 297.

finished concrete work will occur, the molds should have beveled edges, and at salient edges of the finished concrete work, triangular strips should be nailed in the corners of the molds, to produce a beveled edge in the concrete.

To prevent the spreading of the sides of the molds cleats are provided at sufficient intervals. In the case of beams and girders, these are generally secured by nailing. In the case of columns and piers and often in walls the cleats are so notched at the ends that long bolts with washers may be used to hold them in place as shown in Fig. 27. In removing the form the

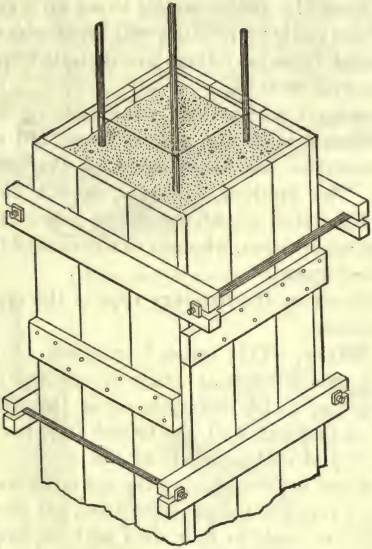

Fig. 27.—Column Forms.

bolts are loosened and the cleats and all are ready to use again. In some cases, particularly in the construction of walls, the cleats are held in place by wire running through the mold, which becomes imbedded in the concrete. In removing the molds the wires are cut and the portions in the concrete are allowed to remain.

The item of molds and centering needed in the erection of reinforced concrete buildings form a considerable part of the cost of construction. Economy in this respect can be affected in designing and planning by making floor panels throughout a

building uniform in size and producing as far as possible repetition of parts, such as piers, walls, etc.

Recently successful attempts have been made to dispense with the erection of timber moulds and centering,—by casting the various members of the construction on the ground and assembling and erecting them in the same manner as wood or steel columns, beams and floors.*

Concrete Mixing.—In all reinforced concrete work the concrete should be mixed mechanically. Satisfactory hand mixing can be obtained and might be resorted to in very small jobs, where it would be uneconomical to set up a machine mixer. But a much more uniform product will be obtained by machine mixing, and most types of mixers are mounted on wheels so as to be easily moved to a job.

Mechanical mixers are either continuous or batch mixers. In the continuous mixers the materials are fed sometimes by hand and sometimes mechanically, and the concrete issues continuously. The product, however, is not likely to be as uniform as in the batch mixer, for in the case of the latter it is under constant supervision, whereas in the case of the former the machine is relied upon.

Of the Batch mixers the rotatory type is the one giving most general satisfaction.

Ransome Mixer.—This mixer † consists of a cylindircal drum, with its axis horizontal, made to revolve about its axis by means of gears. The materials enter through an opening in the center of one end and are mixed together by means of fixed scoops riveted to the inside of the drum. The batch of concrete is dumped without tilting the drum by means of a tilting shute, which receives the concrete from the scoops (Fig. 28).

These mixers are made in four sizes with capacities of 10, 20, 30 and 40 cu. yds. per hour.

McKelvey Mixer.—This mixer ‡ consists of a cylindircal drum rotating axially in which the materials which enter through one end are turned over and over by means of curved "shovels," which pick up the materials and then dump them automatically by gravity as the drum revolves. The mixed concrete is discharged through a long spout without the drum altering its position (Fig. 29).

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* *Eng. News*, July 4th, 1907.

† Made by the Ransome Concrete Machinery Co., N. Y.

‡ Made by McKelvey Concrete Machinery Co., Chicago.

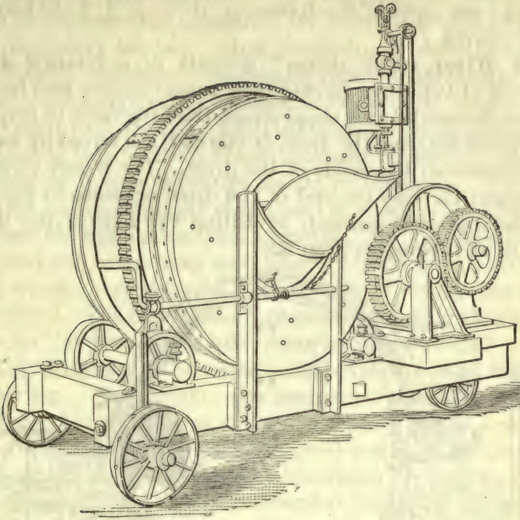

Fig. 28.—Ransome Mixer.

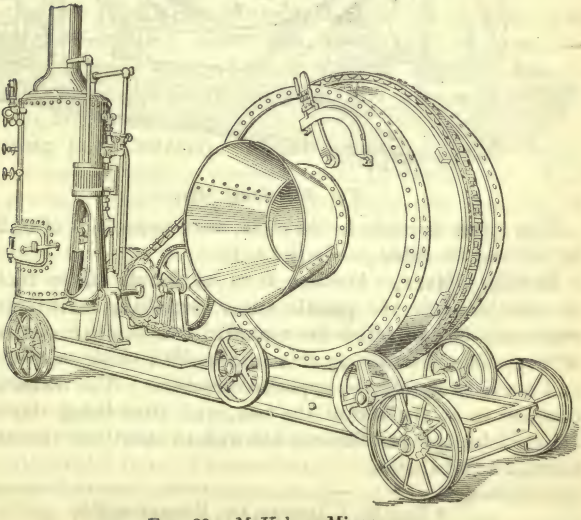

Fig. 29.—McKelvey Mixer.

The mixer is mounted on trucks or steel skids, with or without power, and are built in the following sizes: 1, $\frac{3}{4}$, $\frac{1}{2}$, $\frac{1}{3}$, $\frac{1}{6}$ and $\frac{1}{9}$ yard per charge.

Smith Mixer.—This mixer * consists of a rotating drum in the form of two cones having their bases together and having a common horizontal axis about which they are made to revolve. The materials entering the drum at one end, are mixed by means of fixed blades on the inside of the drum. The mixed concrete is discharged through the end of the drum opposite to that by which the materials entered, by tilting the axis of the drum downward as it revolves (Fig. 30).

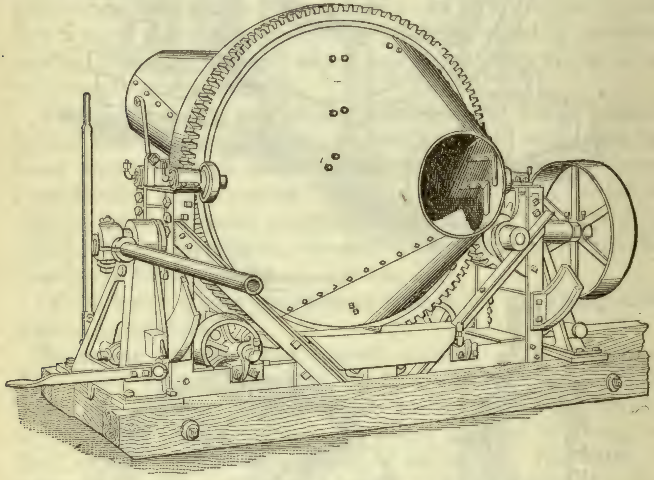

Fig. 30.—Smith Mixer.

This mixer is made in six sizes, with capacities of 9, 20, 30, 39, 46 and 62 cubic yards per hour.

Paddle Mixer.—Another form of batch mixer that may be mentioned is the paddle mixer consisting essentially of a stationary box in which are mounted two shafts each provided with paddles alternating in position with those on the other, and made to revolve in opposite directions. The materials are dumped in at the top of the box, and after being thoroughly churned by the paddles are allowed to drop out through the bottom into barrows.

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* Made by T. L. Smith Co., Milwaukee, Wis.

Charging the Mixers. — In charging, the materials for each batch, carefully measured, are dumped into the mixer and the machinery started. After completing a definite number of revolutions, sufficient to thoroughly mix the ingredients, the concrete is discharged into wheelbarrows or other implements for carrying it to the molds. Each batch should be completed before another is started. To obtain uniform results the number of revolutions in each operation should be the same. It is not well to trust to the judgment of the man in charge of the machine as to when the mixing has been thorough. He should be instructed to count the revolutions each time. A good plan is to attach a gong which rings when the fixed number of revolutions has been completed. The Code of the National Board of Fire Underwriters calls for "at least twenty-five complete revolutions."

Wet Mixture.—The water is introduced during the process of mixing. The amount, also measured, should be such as to produce what is known as a "wet" mixture, that is, a mixture of the consistency of molasses that will readily flow around and thoroughly encase all steel to be imbedded. It may be necessary to vary the amount of water somewhat in placing a large mass of concrete, as in walls, since the water generally works itself upward through the successive layers.

For transporting the concrete from the mixer to the mold, steel wheelbarrows, holding about two cubic feet, are generally employed. A larger vehicle, holding about six cubic feet, is made by the Ransome Machinery Company, and is found very economical in larger work (Fig. 31).

Pouring the Concrete. — Ideal conditions would obtain if the process of placing concrete could be continuous. This is not generally practicable, so it is important that the point at which work is stopped each day shall be so selected and predetermined that the strength of the construction shall suffer least. In smaller buildings, of floor areas not exceeding about 3000 square feet, is should be possible to so arrange the progress of the work that each entire floor construction may be placed in one day, possibly including the columns of the story below. If the columns cannot be done at the same time, they should be poured the day before.

In larger work it is necessary to lay off a certain area to be completed within the time of concreting for the day. Work should not leave off across important beams or girders, but should be made at the middle of slabs or minor floor beams. If any portion

of a floor slab is considered in the calculation for strength of beams or girders, it must be concreted at the same time and as part of such beams or girders.

Ramming.—As soon as the concrete has been poured into the molds, and during the process of pouring, it should be continually rammed to secure complete filling of the molds, density in the finished product, and thorough adhesion to the reinforcement. This ramming should be done, in wet concrete, such as is used for buildings, with a flat steel spatula at the end of a handle long enough for comfortable manipulation. For column work the

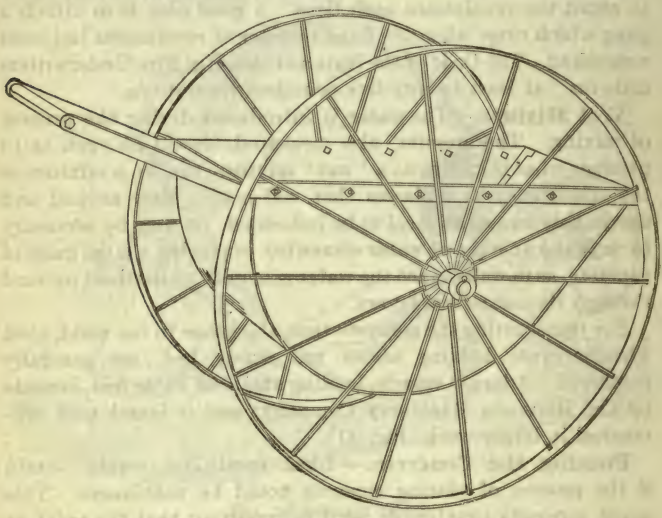


FIG. 31.—Ransome Concrete Cart.

handle is lengthened out so as to reach to the bottom of the forms. Ordinary spades are sometimes employed, and where no special tools are provided rammers are sometimes made of 2″ x 3″ scantlings rounded off at the top end to make a handle.

Where a smooth surface is desired the spatula rammer should be used, particularly at the sides of the molds. The honeycombed appearance that results from improper ramming is difficult to remedy afterward without a patched appearance.

After having been placed the concrete should be kept damp, by sprinkling with a hose, until it has thoroughly hardened.

Removing Forms. — No fixed rule can be given for the removal of the forms, as the time required for the setting of the concrete will vary with the consistency of the mixture, the climatic and other conditions. Numerous failures of reinforced concrete have been attributed to the too early removal of forms. In warm weather concrete will set more quickly than in cold. The setting process may be somewhat accelerated after a day or two, by the removal of the boards forming the sides of beams or girders, leaving in the planks on the under-side and the props supporting them. In cold weather it is well to warm the building during the setting process by means of salamanders.

Finish.—The exposed surfaces of concrete walls are variously treated in attempts to produce a satisfactory appearance. Where no special provision is made, the marks of the lumber used in the forms are almost certain to show, and the lines of demarcation between successive layers are clearly defined. To eliminate these lines, grooves are sometimes purposely formed, by tacking on the sides of the molds triangular or trapezoidal strips that would produce sunk joints in the wall, giving it an appearance resembling dressed stone. The successive layers of concrete are, in such cases, stopped at these lines so that the junction of the two layers is hidden.

In some cases the surface is purposely left rough and scratched like the scratch coat in plastering, and then stuccoed with a neas cement or a rich cement mortar. In this form of finish there it always some danger of the stucco flaking off.

The surface, as it comes from the mold, is sometimes hammer-dressed, or rather picked with a special hammer. This hammer has an edge at right angles to the handle, which edge is so indented as to form a series of points along the edge. This produces a roughened face which in time attains a uniform appearance.

Another method sometimes employed is to remove the forms as soon as the concrete is sufficiently hard, and to rub the surface with a plasterer's float using fine sand between the float and the wall surface and plenty of water.

Floor Finish.—If the floor surfaces are not to be covered with a wooden flooring, a satisfactory finish may be obtained by placing over the surface, before the concrete has had time to set thoroughly, a mortar finish, one to one and a half inches thick, and troweling to get it smooth and level. If the finish is attempted

after the concrete is set, the new and the old will not bond, and there is always danger of flaking off unless the finish itself is of considerable thickness.

A fluid has been put on the market by a Mr. Livingston, of Baltimore, which is said to have some merit in causing the bonding of old and new concrete work. It is applied to the old surfaces just before placing the fresh concrete. The composition is not revealed by the inventor.

The Ransome Concrete Company of New York controls a bonding process for securing a bond between old and new concrete. Tests made by R. W. Hunt and Company seem to substantiate the claim of efficiency. Further information should be obtained from the above mentioned company which controls the patents.

Inspection.—In all reinforced concrete work it is of extreme importance to have competent and thorough inspection or superintendence. The inspector should be familiar with the nature and qualities of the different materials entering into the construction. He should have a knowledge of the underlying principles of the design of reinforced concrete structures, so that he may realize the importance of carrying out all the details, and particularly of placing the reinforcement exactly as planned. He must be sufficiently alert and active to see that the work of the contractor is progressing properly, so that, for instance, work shall not have to be rebuilt because of error in the form.

The materials used in the construction, particularly the cement, should be tested as the work progresses. Cubes of the concrete as used should be made up each day and at the end of seven days should be tested for compression, and if necessary again at the age of twenty-eight days. This record will serve as a guide in the acceptance of the work, or in deciding on the necessity for a load test of the finished structure. Under no circumstances, however, should it replace or serve as an excuse to omit the testing of the cement upon delivery or before acceptance.

In addition to the details discussed in this chapter, that require the attention of the inspector on the work, a few others may be especially mentioned here.

In joining new work with that already in, and which has begun to set, the surface must be thoroughly cleaned and wet. In stopping off work it is good practice where possible, in a urface that will be joined with another, to cast a groove in it, so that when the work is afterward continued, a tongue and groove junction is effected.

All forms or molds must be carefully cleaned out just before pouring concrete. The bottoms of the column molds must be especially watched for this, as shavings, sawdust and even blocks of wood are liable to fall into them unobserved. It is well to leave off a small piece of one side of the column mold at the bottom, for purposes of observation and cleaning, which is closed up just before pouring the concrete.

Great care should be exercised in pouring and ramming concrete in deep molds, such as for columns, walls, etc., to get the molds thoroughly filled at the bottom. In careless work it is not unusual to find in such places very porous concrete, if not large pockets. This is particularly liable to occur when there is considerable reinforcing steel in the construction.

It should be remembered that concrete in setting shrinks. Hollow spaces at the tops of columns are sometimes found due to this cause. As these are not always observable from the outside after the forms are removed, great care should be exercised to guard against them. Therefore, in pouring, the molds should be filled to the top of deep molds to overflowing.

The exact position of the reinforcing steel in the concrete is of such vital importance that particular mention is again made here. In loose bar construction the greatest care must be exercised, in the first place, to have the reinforcement carefully placed, and then to avoid its being shifted out of position by the pouring and the ramming of the concrete.

The reinforcing steel of such systems in which the advantage of attached stirrups is claimed, is often, for convenience in shipping, sent with the stirrups laid flat or close to the main bar. It is intended that in placing them on the job the stirrups shall be turned up to their proper positions. Unless carefully inspected, this is liable to be neglected.

The use of a "unit" type of construction (see page 869) practically obviates these two last mentioned dangers, as the entire reinforcement comes framed together, so that the relative positions of reinforcing rods or bars cannot be changed, and a glance will show whether the "frame" is complete or has been damaged, and when placed in the molds whether it fits or not. In this type of construction the parts are all assembled in the shop from details carefully drawn and checked, in much the same way that steel beams, girders, columns, etc., are fabricated from detailed shop drawings. The work of the inspector or

superintendent on the job is very much simplified, and hence the liability of error reduced to a minimum.

Load Tests.—Load tests on the finished structure should only be resorted to when all reasonable care has been exercised to obtain good results, but, in spite of which, some doubt exists as to the results. Such tests, however, should not be accepted in place of a strict compliance with the specifications.

The architect should know beforehand that his building is correctly designed and is safe, employing, if necessary, an engineer to provide the design.

The contractor should understand at the outset that the structure has been designed for certain purposes and loads, and that the materials specified and the manner of their application are necessary and invariable.

If the contractor furnishes the design, as is sometimes done, (a practice to be condemned), the architect should prescribe in his specifications that such design shall be checked and approved by an engineer appointed by him.

The New York Building Regulations provide for load tests, when found necessary, of three times the gross load for which the construction was designed, under which there should be no signs of distress. This seems unfair and illogical, as a working stress in the steel of about half the elastic limit is permitted. Under these circumstances, the prescribed test load would stress the steel beyond its elastic limit, if the construction is economically designed, and the floor, though showing novisible defects, is nevertheless crippled and unsuited for its working load.

A fair load to be applied in a test would be one and one-half times the working live load plus one-half the weight of the construction. The stresses in the construction would then equal one and one-half times the working stresses assumed in designing. Under these conditions there should not be any evidences of distress, and the deflections should not exceed one-three-hundred-and-sixtieth of the span. The material used for the load test should be so selected and placed that, when uniformly distributed, as required, it will not arch and assist the compressive strength of the beam or floor. Pig iron is a very good material to use. Bricks are more generally available, but must often be piled very high to get the required load, consuming much time and labor in making the test. Iu using bricks they should be set in vertical piles with spaces of two or three inches between them, thus avoiding all arching of the load.

CHAPTER XXV.

ROOF TRUSSES.

TYPES OF WOODEN AND STEEL TRUSSES—THEIR LIMITATIONS AND REQUIREMENTS.

WHENEVER it is required to roof a hall, room, or building with a clear span of more than thirty feet, it is generally necessary to use one or more trusses to support the roof and ceiling.

Definitions.—By the term "truss" as used in this and the following chapters, the author means "a framework supported only at the ends (or in the case of a cantilever, near the centre), and so designed that it cannot suffer distortion without either crushing or pulling apart one of the members of which it is composed, and will exert only a *vertical* pressure on the walls." A true truss does not depend upon the rigidity of the joints to maintain its equilibrium.

A roof truss is a form of truss designed for the especial purpose of supporting a roof, although it may also support the ceiling below and perhaps a gallery or one or more floors.

Roof trusses are or should be designed upon the same principle as other trusses, the difference between a roof truss and a bridge truss being due to the difference in the shape of the truss and the character of the load to be supported, rather than in the mechanical principles involved.

By *wooden trusses* is meant trusses built principally of wood, but having iron or steel rods for some of the tension members, the term being used in distinction from trusses built wholly of steel. The term "combination truss" is also sometimes used to designate such trusses.

A *member* of a truss is any straight or curved piece of wood, iron, or steel which connects two adjacent joints of a truss, and which is essential to the stability of the truss. The term "piece" will also frequently be used to designate a particular member. Every member of a true truss acts either as a strut or a tie.

A *tie* is a member that is subject to tension; *i.e.*, a pulling stress.

A *strut* is a member that is subject to a compressive stress.

In wooden trusses, the struts are always made of wood, but the ties may be of wood, wrought iron, or steel.

A *tie-beam* is a tie which is also subject to a transverse strain; in wooden trusses the principal tie is usually called the tie-beam, even when it has no transverse strain except that due to its own weight.

A *strut beam* is a strut that is also subject to a transverse strain. In wooden trusses, the horizontal struts are sometimes termed "straining beams."

The *joints* of a truss are the points where three or more members meet, although two of the members may be formed of the same piece of material.

Purlins are horizontal beams, sometimes trussed, extending from truss to truss to support the rafters or ceiling joists.

Types of Wooden Trusses.

The simplest truss that can be built is that shown by Fig. 1, which consists only of two struts or rafters and a tie-beam.

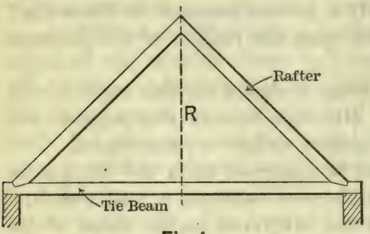


Fig. 1

As the unsupported length of a strut, on account of economy, should not exceed 12 feet, such a truss is not suitable for spans exceeding 20 to 24 feet, and even for a span of 20 feet there should be a centre rod, as shown by the dotted line *R*, to support the tie-beam. To utilize this truss for a greater span than 24 feet, it will be necessary to brace the rafters from the foot of the centre rod as shown by Fig. 2. This gives us the *king-rod truss*, the modern type of the old-fashioned king-post truss shown by Fig. 3, which was built wholly of wood except for the iron straps at *S* and *P*.

When the tie-beam supports a ceiling or attic floor, rods should be inserted at *R R*, Fig. 2, to support the load on the tie-beam. By increasing the number of rods and braces, as in Figs. 4 and 5, this type of truss may be used for spans up to 64 feet, and even for greater spans, but it is not an economical

type when the span exceeds 60 feet, on account of the increased length of the centre braces and rods. When there is no

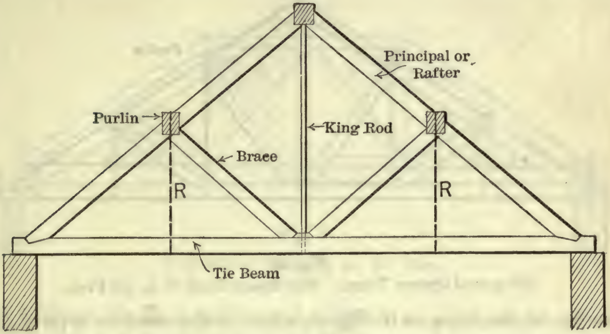
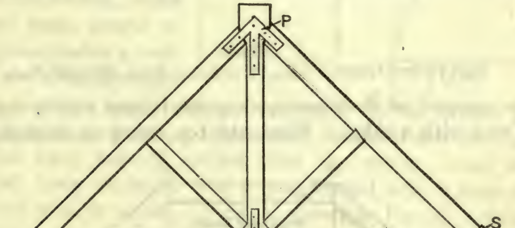


Fig. 2
King-rod Truss. For Spans up to 36 Feet.

load on the tie-beam the rods *R R*, Figs 4 and 5, may be omitted.

Note.—The names given to the trusses shown by Figs. 4 and 5 are original with the author. These trusses are sometimes called queen trusses, but as the term queen truss commonly means a truss such as is shown by Fig. 6, the author has pre-



FOR SPANS FROM 25 TO 35 FT.
Fig. 3

fixed the words "six-panel" and "eight-panel" to give a more definite meaning to the name of the truss.

The rise of the rafter in any of the trusses, Figs. 1–5, should never be less than 6 ins. in 12 ins., or 26½°, and a ⅓ pitch, or a

rise of 8 ins. in 12 is generally the most economical. When the span exceeds 36 feet, it is generally more economical to cut off

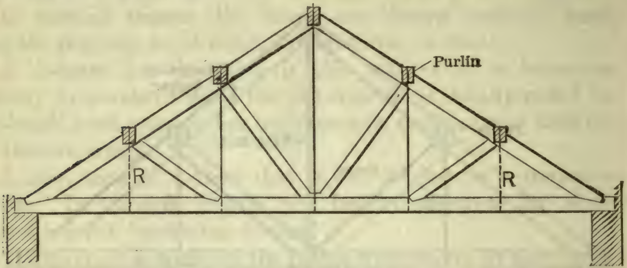


Fig. 4
Six-panel Queen Truss. For Spans from 36 to 50 Feet.

the top of the truss as in Fig. 6, which is the modern type of the ancient queen-post truss. This truss is quite frequently used

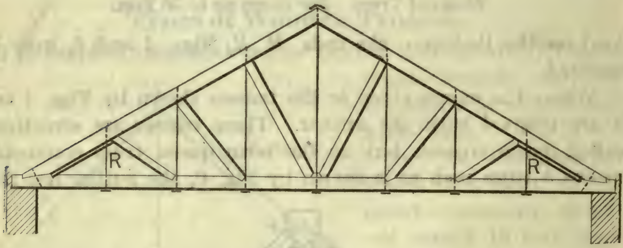


Fig. 5
Eight-panel Queen Truss. For Spans from 48 to 60 Feet.

for the support of deck roofs, although it may also be used for a pitch roof with a ridge. When the top chord or straining beam

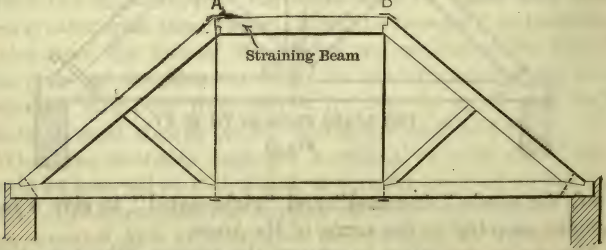


Fig. 6
Queen-rod Truss. For Spans from 30 to 45 Feet.

is more than 12 feet long, the size of the chords may be considerably reduced by using a centre rod and a pair of braces as shown in Fig. 7. The centre rod will be especially needed if

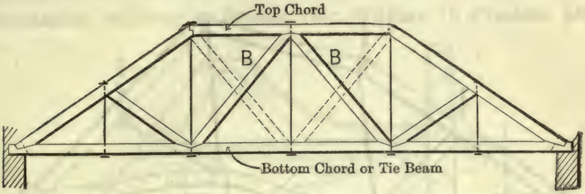


Fig. 7
For Spans from 40 to 52 Feet.

the bottom chord or tie-beam is subject to a transverse strain. The centre rod should never be used, however, unless the braces, *B B* are also added.

Counter Braces.—The truss shown in Fig. 6 differs from the trusses shown in Figs. 1 to 5 in one very important respect. The trusses 1 to 5 are composed of *triangles*, while the centre panel of truss 6 is a *rectangle.* Now a triangle cannot be changed in shape without lengthening or shortening one side, but a rectangle can be distorted without changing the length of the sides. Thus in Figs. 8 and 9 the corresponding sides have the same length in both figures, hence a rectangle is not a rigid shape.

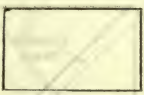


Fig. 8


Fig. 9

For this reason a truss having a rectangle for the centre panel and built with pin-joints would not be stable if one side of the truss was more heavily loaded than the other. Thus if the queen-rod truss shown in Fig. 6 was loaded with 6 tons at *A*, and 3 tons at *B*, it would collapse as shown in Fig. 10 unless the tie-beam was large enough to resist the pull from the rod. To counteract this tendency to collapse a brace should be placed in the centre panel inclined downwards from the side that is most heavily loaded.

Thus, Fig. 11 shows the proper construction for a queen-rod truss loaded at one side only, or more heavily loaded on the left-hand side than on the right. It will be seen that this truss is composed entirely of triangles.

In practice, the weight of a roof is generally uniform on both sides, the only variation in the loading being that due to wind and snow. For trusses not more than 36 feet span, and one-third pitch, the tie-beam will generally possess sufficient stiffness

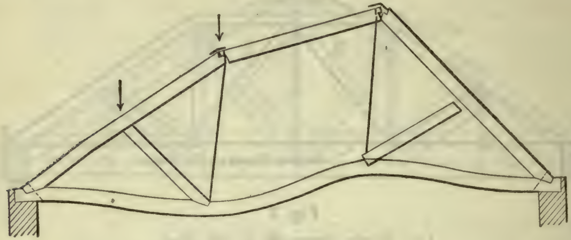


Fig. 10

to resist the tendency of the unequal pressure of wind or snow to distort the truss; but for larger spans, and for a pitch of 45°, *two braces* should be inserted in the centre panel of queen-rod trusses, as one side may receive the greater pressure at one time, and the other side at another time. Braces put in to resist the effect of unequal loading are called *counter braces.* Under a

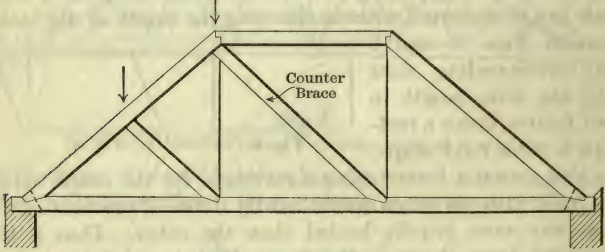


Fig. 11

uniform dead load counter braces receive no stress whatever. *Every truss with horizontal top and bottom chords should be provided with counter braces, whenever there is any possibility of a material variation in the loading.*

When such trusses support a floor, they should always have counter braces, because a floor may be heavily loaded at one point, while the rest of the floor may have no load at all. Thus,

if the truss shown by Fig. 7 supported a floor, counter braces should be inserted as shown by the dotted lines.

Fig. 12 shows an ornamental queen-post truss supporting a portion of the roof of the Massachusetts Charitable Mechanics' Association building in Boston (Mr. William G. Preston, archi-

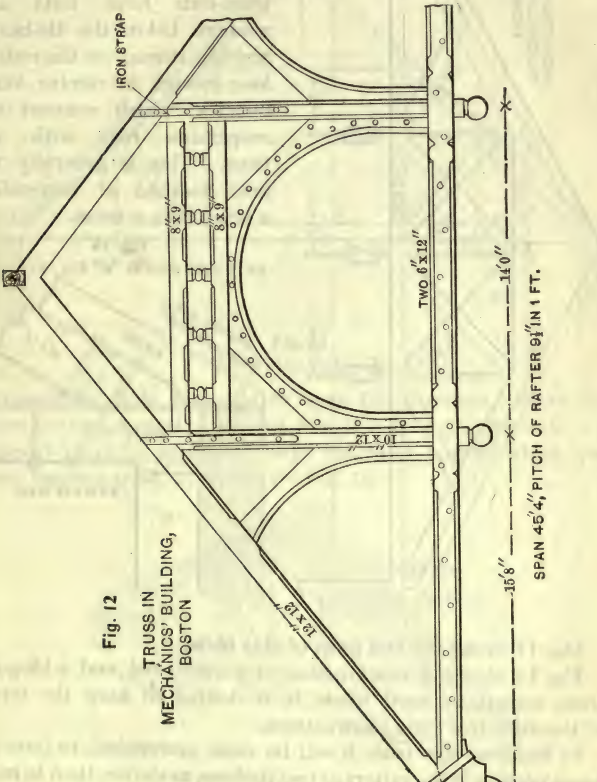


Fig. 12

tect). The members, which are of Georgia pine, were worked from timbers of the dimensions given. In this truss posts are used instead of rods, being bolted and tenoned to the tie-beam and secured to the rafters by iron straps.

The curved ribs take the place of counter braces.

Fig. 13 shows a queen truss from the Museum of Fine Arts, St. Louis, Mo. (Messrs. Peabody & Stearns, architects), which supports the floor below by means of three rods. The truss-rods have nuts and washers below the tie-beam, and the thread on the rods is long enough to receive turnbuckles which connect the suspension rods with the truss. This is generally the best method of suspending a floor from a truss.

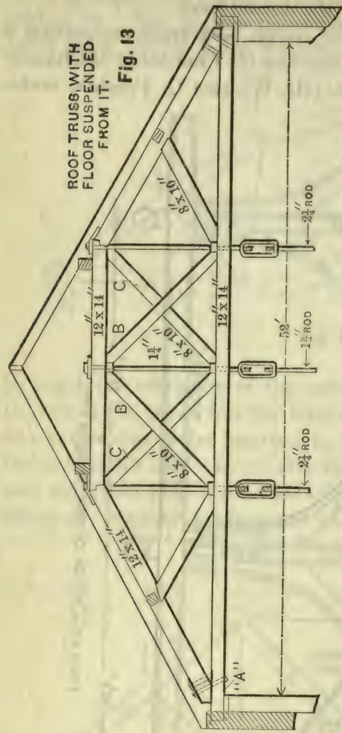


Fig. 13

ROOF TRUSS, WITH FLOOR SUSPENDED FROM IT.

Fig. 14

DETAIL OF JOINT "A" FIG 13.

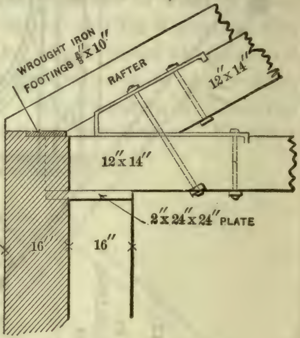


Fig. 14 shows the end joint of this truss.

Fig. 15 shows a combination of a queen-rod and a king-rod truss, sometimes used where it is desired to keep the centre of the attic free from obstructions.

In building this truss it will be more economical to form the lower portion of the rafters of two timbers, as shown, than to make it of one size for the full length. This construction also allows of making a good joint at *B*. What has been said in regard to counter braces in queen-rod trusses applies also to this truss, although with this truss the continuous rafter aids very materially in resisting distortion from wind pressure, so that for ordinary construction and for spans not exceeding 40 feet it will be perfectly safe to omit counter braces.

Manner of Supporting the Common Rafters—Purlins.—Before describing other types of trusses, it may be well to consider the manner of supporting the common rafters by the trusses.

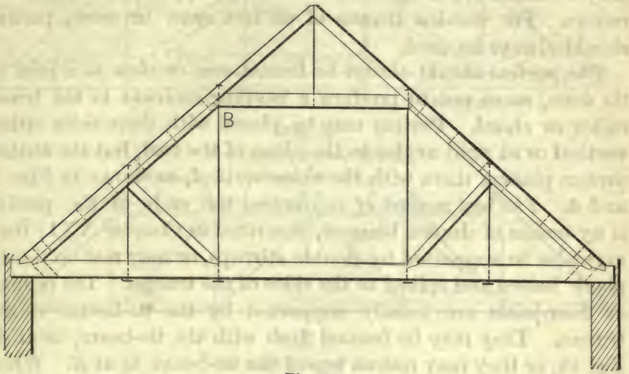


Fig. 15
For Spans up to 42.

Occasionally it is desirable to span the common rafters from truss to truss, but as a general rule it is better construction to support them by means of large beams or purlins which span from truss to truss, as shown by Fig. 16.

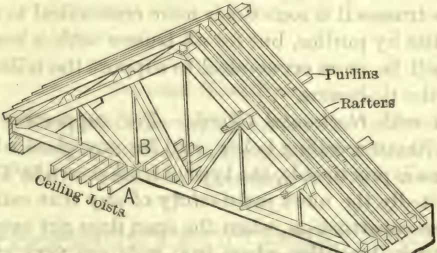


Fig. 16

The trusses can be designed so that the purlins need not be more than 10 feet apart, and very often not more than 6 or 8 feet apart, so that the common rafters need not be more than $2'' \times 4''$ or $2'' \times 6''$ in cross-section while the trusses may be spaced 12, 14, or 16 feet on centres. As a rule a spacing of about 14 ft. for the trusses, and of 9 ft. 6 in. for the purlins, will be found most

economical. Another advantage in the use of purlins is that where the purlins are placed at the truss joints no cross-strain is brought on the truss rafters or chords, and hence the latter may be made much lighter than if they supported the common rafters. For wooden trusses of 60 feet span or more, purlins should always be used.

The purlins should always be located over or close to a joint of the truss, so as not to produce a bending-moment in the truss-rafter or chord. Purlins may be placed with their sides either vertical or at right angles to the plane of the roof, but the author prefers placing them with the sides vertical, as shown in Figs. 2 and 4. *The best method of supporting* the ends of the purlins is by means of duplex hangers, described in Chapter XXI; they may also be supported by double stirrups, or may rest on 3-inch plank bolted and spiked to the sides of the trusses. The ceiling- or floor-joists are usually supported by the tie-beams of the trusses. They may be framed flush with the tie-beam, as at *A*, Fig. 16, or they may rest on top of the tie-beam as at *B*. When the joists are used to support an attic floor it will be better to place them on top of the tie-beam. There is no particular objection to imposing a transverse strain on a tie-beam, as the tension in the beam tends to straighten it, and the cross-section of a wooden tie-beam must always be made considerably larger than would be required to resist the direct tension. In the case of scissors trusses it is sometimes more economical to support the ceiling-joists by purlins, but for all trusses with a horizontal tie-beam it will be more economical to support the ceiling- or floor-joists by the tie-beams.

Trusses with Horizontal Chords.—For supporting flat roofs, with or without a ceiling below, and for any place where a horizontal truss is practicable, the type of truss shown by Figs. 17 to 19 is undoubtedly the most satisfactory of any that can be devised for wooden construction, when the span does not exceed 80 feet, and except in localities where iron rods are very expensive it will be as economical as any. In this work the name "Howe truss" will be given to trusses of this type, as the truss is an adaptation to building construction of the Howe bridge truss. The term "horizontal truss" is also sometimes applied to trusses of this type. Trusses of this type can be made strong enough for spans up to 150 feet, but when the span exceeds 100 feet it will probably be cheaper to use some other truss.

When a Howe truss is placed longitudinally of a flat roof,

the top chord may be given the same inclination as the roof, so as to support the rafters without blocking, as shown by

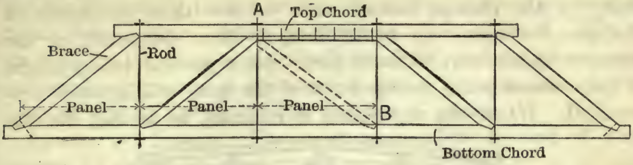


Fig. 17
Five-panel Howe Truss.

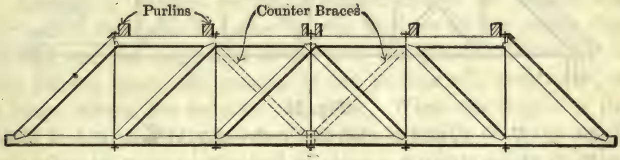


Fig. 18
Six-panel Howe Truss.

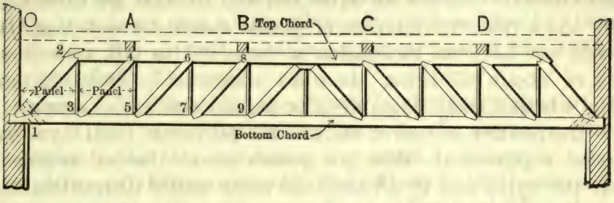


Fig. 19
Ten-panel Howe Truss.

Fig. 20. For deck roofs the top chord may be inclined upwards toward the centre, to conform to the shape of the roof, as shown by Fig. 21. For a deck and mansard roof the centre panels

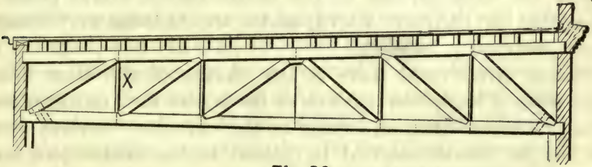


Fig. 20

should have counter braces, as shown in Fig. 21, to resist the wind pressure against the sides of the roof, and any unequal distribution of snow.

Rules to be Observed When Designing a Howe Truss—Height.—The height of the truss, always measured from *centre to centre* of the chords, should *never be less* than one-ninth of the span for spans up to 36 feet, or than one-tenth of the span for spans from 40 to 80 feet. As a general rule a height of from one-seventh to one-sixth of the span will be most economical. When the top chord is inclined, as in Fig. 20, the

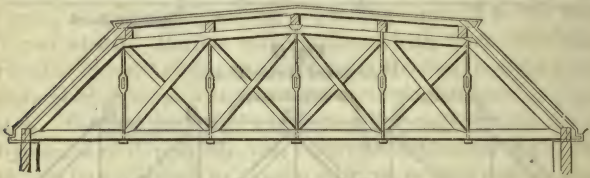


Fig. 21

height at *X—i.e.*, at the shortest rod—should not be less than the limit given above.

Number of Panels.—A panel is the space between two adjacent rods or between an outer rod and the end joint (see Fig. 17). As a rule, the number of panels should be such that the braces will have an inclination of from 36° to 60°, an inclination of about 45° being the most economical. It is not material whether there be an even or an odd number of panels.

If the position of one or more of the purlins is fixed by some special requirement, then the panels should be so arranged that the upper end of a brace will come under the purlin, and that the inclination of none of the braces will be less than 36°.

Although it is generally better to have the truss symmetrical about the centre, it is not absolutely necessary, nor is it necessary that the panels be of uniform width. When the truss is *not symmetrically loaded*, however, it may be necessary to reverse the brace in one of the centre panels. This point is considered in Chapter XXVI, under the heading of "Trusses Unsymmetrically Loaded."

Counter Braces.—If there is any chance of the truss being more heavily loaded on one side of the centre than on the other, counter braces—that is, braces in the opposite direction from the regular braces—should be placed in the centre panels as shown by dotted lines in Fig. 18.

When a load of much magnitude is placed on one side of a truss having an *odd number* of panels, without a corresponding load on the other side, a brace *will always be required in* the

centre panel, and the brace should incline *downward* from the more heavily loaded side. Thus, if the truss shown by Fig. 17 were more heavily loaded to the left of the centre than to the right, then a brace would be required from *A* to *B*. When the load on the truss is practically uniform, counter braces are not necessary, nor is it necessary to put a brace in the centre panel of a truss having an odd number of panels.

Spacing of Trusses.—The most economical spacing of the trusses, all things considered, will usually be from 12 to 16 feet for spans up to 60 feet, and from 14 to 20 feet for greater spans.

Spacing of Purlins.—Purlins should always be placed *over the end of the braces* and close to the washers on the rods; they should also be spaced so as to give the greatest economy in the rafters, hence the spacing of the purlins will determine, to a large extent, the number of panels. When the height of the truss is not more than one-ninth or one-tenth of the span, it will often be more economical to place a purlin over every other joint, as in Fig. 19.

Bearing on Wall or Post.—The point where the centre lines of the end brace and of the tie-beam intersect should always come over the support, and generally at least 6 ins. beyond the inner face of the wall.

Stresses.—The strain in the chords is always greatest at the centre of the truss, diminishing towards the supports, while the stress in the rods and braces is greatest at the ends.

Table of Dimensions for Howe Trusses.—For symmetrical trusses having panels of uniform width and uniformly loaded, the stresses in the different parts will be proportional to the span, number of panels, height of truss, spacing of trusses, and the load per square foot. It is therefore possible to prepare tables giving the dimensions of the parts for such trusses. Table I., computed by the author, gives the dimensions for six-panel trusses for heights of one-sixth and one-eighth of the span, and for three different spacings.

These dimensions are *for a flat roof* of tin, sheet iron or composition, and for a snow load of 16 pounds per square foot, which is equivalent to about 24 ins. of light, dry snow; *also for a lath and plaster ceiling* supported by the tie-beam; the chords and braces being of *Norway pine* and the rods of wrought iron.

These dimensions apply only when the rafters are supported on purlins placed at the upper joints, as in Figs. 18 and 19. When the rafters rest on the top chord, as in Fig. 20, the dimen-

sions of the latter must be greatly increased, and special calculations should be made therefor.

The dimensions given in the table may be used for trusses having a greater height than that given, but *not for trusses with*

TABLE I.—DIMENSIONS FOR SIX-PANEL HOWE TRUSSES, SYMMETRICALLY LOADED.

Timber, Norway pine, Oregon pine, or Eastern spruce.

Span.	Distance apart, C to C.	Total height.	Top chord.	Bottom chord.	Braces. A.	Braces. B.	Braces. C.	Rods (not upset). D.	Rods (not upset). E.	Rods (not upset). F.
Ft.	Ft.	Ft. Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.
36	12	6 7	6× 6	6× 8	6× 6	6× 4	6× 3	1⅛	¾	⅝
		5 2	6× 8	6× 8	6× 6	6× 6	6× 4			
	15	6 8	6× 8	6× 8	6× 6	6× 4	6× 3	1¼	⅞	⅝
		5 2	8× 8	8× 8	8× 6	6× 6	6× 4			
	18	6 8	6× 8	6× 8	6× 8	6× 6	6× 4	1¼	⅞	⅝
		5 2	8× 8	8× 8	8× 8	6× 6	6× 4			
42	12	7 7	8× 6	8× 8	8× 6	8× 4	6× 4	1¼	⅞	⅝
		5 11	8× 8	8× 8	8× 6	8× 5	8× 4			
	15	7 8	8× 8	8× 8	8× 6	8× 5	6× 4	1⅜	⅞	¾
		5 11	8× 8	8× 8	8× 8	8× 6	8× 4			
	18	7 8	8× 8	8× 8	8× 8	8× 6	8× 4	1½	1	¾
		6 1	8×10	8×10	8× 8	8× 6	8× 4			
48	12	8 8	8× 8	8× 8	8× 8	8× 6	8× 4	1⅜	⅞	¾
		6 8	8× 8	8× 8	8× 8	8× 6	8× 4			
	15	8 8	8× 8	8× 8	8× 8	8× 6	8× 4	1⅜	1	¾
		6 10	8×10	8×10	8× 8	8× 6	8× 4			
	18	8 8	8× 8	8× 8	8× 8	8× 6	8× 4	1½	1	¾
		6 10	8×10	8×10	8×10	8× 6	8× 4			
54	12	9 8	8× 8	8× 8	8× 8	8× 6	8× 4	1⅜	⅞	¾
		7 6	8× 8	8×10	8× 8	8× 6	8× 4			
	15	9 8	8× 8	8× 8	8× 8	8× 6	8× 4	1½	1	¾
		7 7	8×10	8×10	8× 8	8× 6	8× 4			
	18	9 10	8×10	8×10	8×10	8× 8	8× 6	1⅝	1⅛	¾
		7 7	10×10	10×10	10× 8	8× 8	8× 4			
60	12	10 9	8× 8	8×10	8× 8	8× 6	6× 6	1⅜	1	¾
		8 4	8×10	8×10	8×10	8× 6	8× 4			
	15	10 10	8×10	8×10	8×10	8× 6	6× 6	1½	1⅛	¾
		8 4	10×10	10×10	10× 8	10× 6	8× 4			
	18	10 10	10×10	10×10	10× 8	10× 6	8× 6	1¾	1⅛	¾
		8 4	10×10	10×10	10×10	10× 6	8× 6			
70	12	12 6	8×10	8×10	8×10	8× 6	6× 6	1½	1	¾
		9 7	10×10	10×10	10× 8	10× 6	8× 6			
	15	12 6	10×10	10×10	10× 8	10× 6	8× 6	1¾	1⅛	¾
		9 9	10×12	10×12	10×10	10× 8	10× 6			
	18	12 6	10×10	10×10	10×10	10× 6	8× 6	1⅞	1¼	⅞
		9 9	10×12	10×12	10×12	10× 8	10× 6			
80	12	14 2	10×10	10×10	10×10	10× 6	8× 6	1⅝	1⅛	¾
		10 10	10×10	10×10	10×10	10× 6	8× 6			
	15	14 2	10×10	10×10	10×10	10× 8	8× 6	1⅞	1¼	⅞
		11 0	10×12	10×12	10×10	10× 8	10× 6			
	18	14 4	10×12	10×12	10×12	10× 8	8× 6	2	1⅜	1
		11 1	10×12	10×14	10×12	10× 8	10× 6			

a less height, as the less the height the greater will be the stresses in the chords and braces. When the conditions of load, span, height, and spacing are not exactly as given above and in the table, *the stresses should be determined* and the parts of the truss proportioned accordingly; but even in such cases the table will serve somewhat as a check upon the computations.

Lattice Trusses.—In localities where lumber is cheap, and iron or steel rods quite expensive, the lattice truss (Fig. 22) will often be the most economical for supporting flat roofs.

This type of truss was designed by Ithiel Towne for bridges long before iron was used in this country for such work. Several railroad bridges were built on this principle, and the truss has proved very efficient in supporting loads. The principal objection to the truss, from a mechanical standpoint, is that the truss cannot be tightened up, and the joints are not as satisfactory as in a Howe truss.

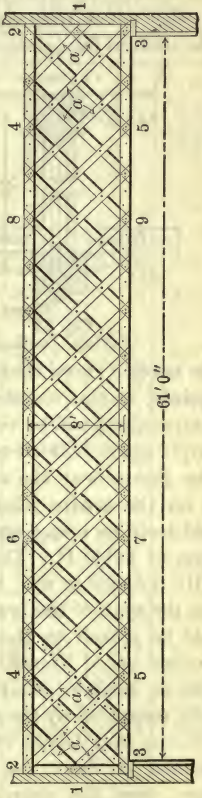


Fig. 22
Lattice Truss.

Proportions and Construction.—The height of a lattice truss, *measured between the centre lines of the chords,* should be from one-eighth to one-sixth of the span, and the braces should be placed at an angle of about 45 degrees. When laying out a lattice truss, the first step should be to determine the height, and then the number of spaces between the joints in the top and bottom chords. To find the number of spaces, multiply the span by two, and divide by the height, using the nearest whole number. Thus, if the span is 60 feet, and the height 8 feet, there should be $\frac{2\times 60}{8}=15$ spaces. If the height is 10 feet, there should be 12 spaces. The truss shown in Fig. 22 has 16 spaces.

Having determined the height and number of spaces, fix the centre of the end joints, and divide the distance between into the number of spaces determined upon, thus fixing the position of the braces. The chords should be built of four thicknesses of plank, two on each side of the truss, and breaking joint opposite their centres, using as long planks for the tie-beam as can be obtained. At the ends, vertical planks should be cut between the chords, on each side of the bracing, to act as posts. The braces should be bolted to the chords and end-posts, and also to each other where they cross. A goodly number of spikes should also be used in the joints, as indicated in Fig. .24

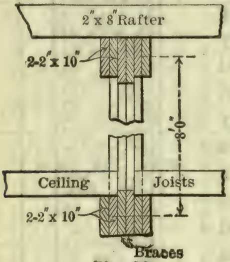


Fig. 23
Vertical Section.

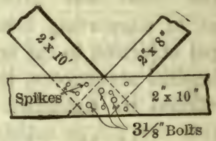


Fig. 24
Detail of Joint B.

The bottom chord should also be bolted every two feet between the joints, as this member is in tension. The top chord, being in compression, will be tied sufficiently by the bolts at the joints, and by a short bolt on each side of each butt-joint. The strain on the joints near the ends of the truss will be much greater than on the centre joints. The first three joints at each end should have as many and as large bolts as are given in the last column of Table II. The bolts in the next three joints may be slightly reduced in size, and those in the centre joints still more. When the span of the truss exceeds 40 feet, short pieces of plank should be spiked to the end braces *a a* fitting tightly between the other set of braces, to give them additional strength. It should be kept in mind that the strength of a lattice truss is usually measured by the strength of the joints.

Stresses in a Lattice Truss.—The stresses in a lattice truss are computed in about the same way as those in a riveted plate girder; thus the chords are assumed to resist the bending-moment, and the braces the shearing-stress.

The tension or compression in the chords is greatest at the centre, while the stress in the braces is greatest at the ends.

Under a uniformly distributed load, the maximum stress in the chords may be found by *multiplying the total load by the span and dividing by* 8 times the height, both in feet.

The stress in each of the end braces *a a a* when the angle of inclination approximates 45°, will be one-sixth of the total load, multiplied by 1.4.

The following table gives the dimensions for lattice trusses,

TABLE II.—DIMENSIONS FOR LATTICE TRUSSES OF FIRST QUALITY WHITE PINE OR SPRUCE.

To support a gravel roof and plastered ceiling, allowing 20 pounds per square foot for snow.

Span.	Spacing of trusses.	Height out to out.		No. of spaces.	Size of bottom chord.	Size of top chord.	End braces.	Inner braces.	Bolts in joints 1—5. See Fig. 22.
Ft.	Ft.	Ft.	Ins.						Inch.
40	12	5	6	16	4—2× 6	4—2× 6	2× 6	2× 6	2—1
		7	2	12	4—2× 6	4—2× 6			
	14	5	7	16	4—2× 6	4—2× 8	2× 6	2× 6	3—⅞
		7	3	12	4—2× 6	4—2× 8			
	16	5	8	16	4—2× 8	4—2× 8	2× 8	2× 6	3—1
		7	4	12	4—2× 8	4—2× 8			
50	12	6	8	16	4—2× 8	4—2× 8	2×10	2× 8 and 2× 6	3—1
		8	8	12	4—2× 8	4—2× 8			
	14	6	8	16	4—2× 8	4—2× 8	2×10	2× 8 and 2× 6	3—1⅛
		8	8	12	4—2× 8	4—2× 8			
	16	6	9	16	4—2× 8	4—2×10	2×10	2× 8 and 2× 6	3—1⅛
		8	8	12	4—2× 8	4—2× 8			
60	12	8	4	16	4—2×10	4—2×10	2×10	2× 8	3—1⅛
		10	10	12	4—2×10	4—2×10			
	14	8	4	16	4—2×10	4—2×10	2×10	2× 8	3—1⅛
		10	10	12	4—2×10	4—2×10			
	16	8	4	16	4—2×10	4—2×10	2×10	2× 8	3—1¼
		10	10	12	4—2×10	4—2×10			
70	14	9	5	16	4—2×10	4—2×12	2×10	2× 8	1—2¼
		12	4	12	4—2×10	4—2×10			
	16	9	5	16	4—2×10	4—2×12	2×10	2× 8	1—2⅜
		12	4	12	4—2×10	4—2×10			
	18	9	6	16	4—2×12	4—2×12	2×10	2× 8	1—2½
		12	6	12	4—2×12	4—2×12			
80	14	11	0	16	4—2×12	4—2×12	2×10	2× 8	1—2¾
		14	0	12	4—2×12	4—2×12			
	16	11	2	16	4—2×14	4—2×14	2×12	2×10 and 2× 8	1—3
		14	0	12	4—2×12	4—2×12			
	18	11	2	16	4—2×14	4—2×14	2×12	2×10 and 2× 8	1—3
		14	1	12	4—2×12	4—2×14			

Uprights at end same size as end braces.

built as shown in Fig. 22, for five different spans, and different spacings and heights, which will cover nearly all of the conditions under which these trusses should be used. In localities where a fall of snow two feet in depth is liable to occur, these dimensions should be increased.

Referring to Fig. 22, it may be stated that each chord of the truss is built of four 2×10's in 10 and 20 foot lengths, the braces *a a a* are 2×10 and the other braces 2×8. The joints at 1, 2, 3, 4, and 5 have three 1⅛-inch bolts, the joints between 6 and 8 and 7 and 9 have two ⅞-inch bolts, while the other joints have two 1-inch bolts. There should also be two ⅞-inch bolts in the tie-beam, in each space between the joints, to assist in transmitting the tension from one plank to the other.

Wooden Trusses with Raised Tie-beams.—All of the trusses thus far described have horizontal tie-beams, which are the most desirable as well as the most economical, whenever the conditions will permit.

In roofing churches, public halls, etc., a raised ceiling is often desired in order to give greater height to the room, without increasing the height of the walls.

Scissors Trusses.—For such roofs some form of the scissors truss (so named from its resemblance to a pair of scissors) is most often used. When correctly designed, with members of the proper size, and with the joints carefully proportioned to the stresses, the scissors truss makes a very good truss for supporting the roof over halls and churches, up to a span of 48 feet, but above that they should be used with much caution, as the stresses become very great, and the joints difficult to make.

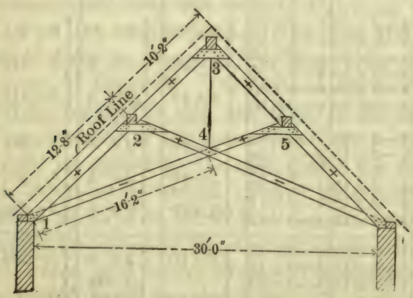


Fig. 25

Figs. 25 to 30 show different forms of the truss adapted to different spans and roof pitches.* None of these trusses will exert a horizontal thrust

* The dimensions given in these figures are for use in computing the roof loads in the examples given in Chapter XXVI.

if the members are of ample size and the joints properly made. The members having a + sign on or close to them are in compression, while the others are in tension.

The members indicated by a single line should be rods (except in the case of tie-beams).

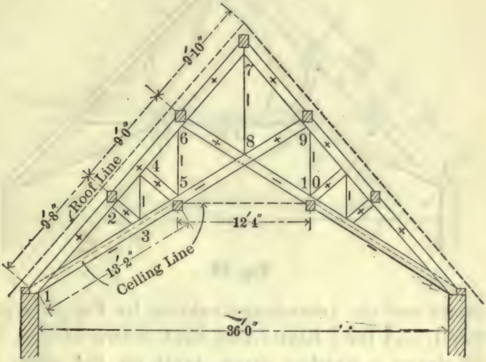


Fig. 26

Fig. 25 shows the simplest form of the scissors truss, which is suitable for spans up to 30 feet. When the span exceeds

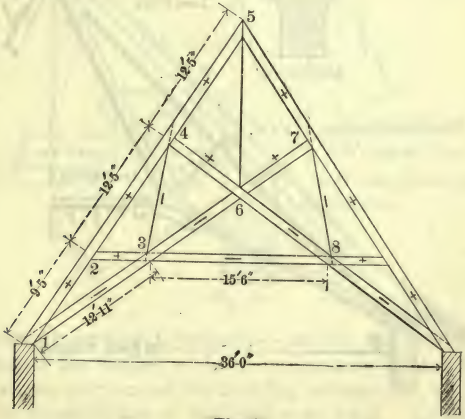


Fig. 27

30 feet, it will be more economical to use two purlins on each side, to support the common rafters, and additional supports

will generally be required by the tie-beams which will call for the additional rods and braces shown in Fig. 26.

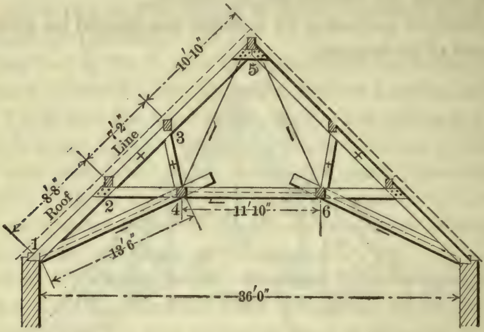


Fig. 28

For a steep roof the arrangement shown by Fig. 27 is generally best adapted, and for a flatter roof that shown by Fig. 28.

Fig. 29 shows a finished truss, built on the same lines as

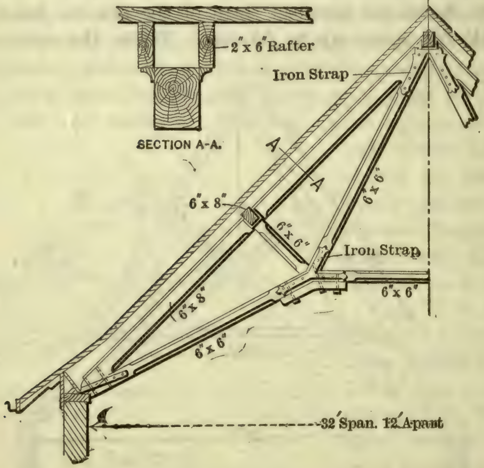


Fig. 29

Fig. 28, but with only one purlin. The truss shown by Fig. 30 is similar to that shown by Fig. 27, with the peak cut off, and

a straining beam placed between the upper ends of the principals, and the centre rod omitted. For spans exceeding 36 feet, Fig. 30 gives a more economical truss than Fig. 27; this truss can also be used where the roof is hipped. With this truss it will be better to use ceiling purlins to support the ceiling joists than to span them from truss to truss.

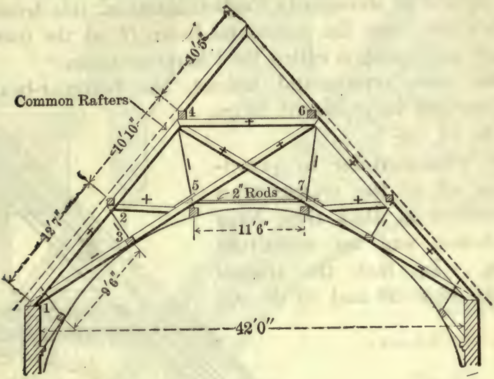


Fig. 30

Fig. 31 shows the best way of making the joints at 5 and 7 of Fig. 30.

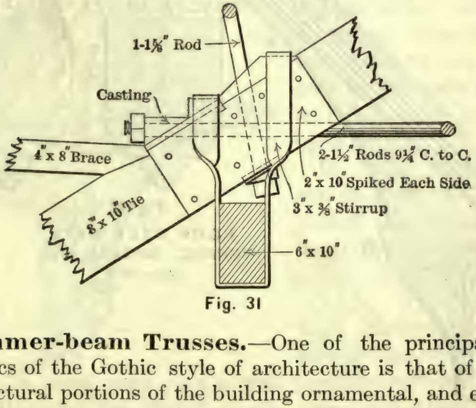


Fig. 31

Hammer-beam Trusses.—One of the principal characteristics of the Gothic style of architecture is that of making the structural portions of the building ornamental, and exposing the whole construction of the edifice to view. As the pointed arch and steep roof were developed the roof truss became an important feature in the ornamentation of Gothic halls

and churches. The trusses of this period were built almost entirely of wood, and generally of very heavy timbers, to give the appearance of great strength.

One of the most common types of these Gothic trusses, and also the most ornamental, was that of the "Hammer-beam truss," which is still often used in Gothic churches.

Figs. 32 and 33 show early English forms of this truss, which takes its name from the horizontal beam *H*, at the foot of the principal, and which is called the "hammer-beam."

In the more ornamental trusses, the hammer-beam was usually carved to represent kings or angels.

These trusses differ in principle from all of the trusses thus far described in that they have no tie-beam, and no substitute for one. In fact the trusses shown by Figs. 32 and 33 do not come within the scope of the definition for a truss given at the beginning of the chapter. Although the rafters or prin-

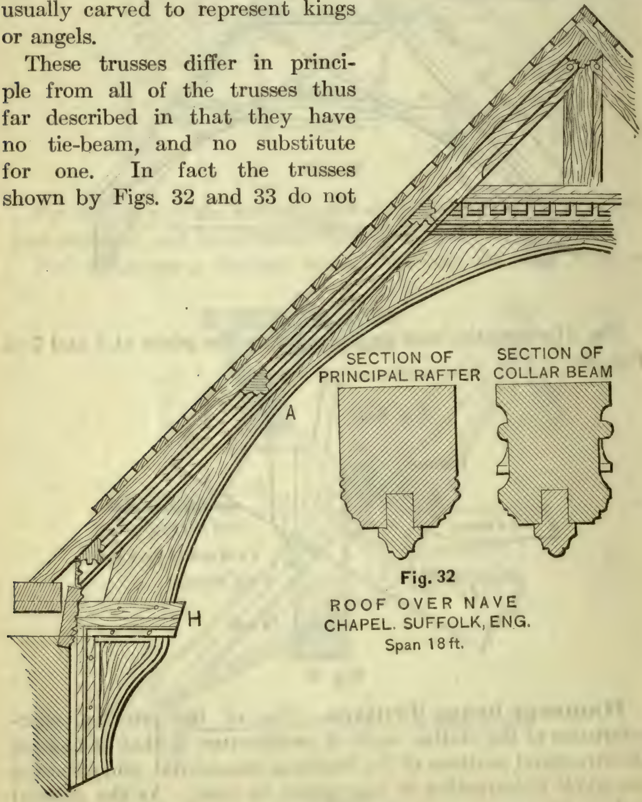
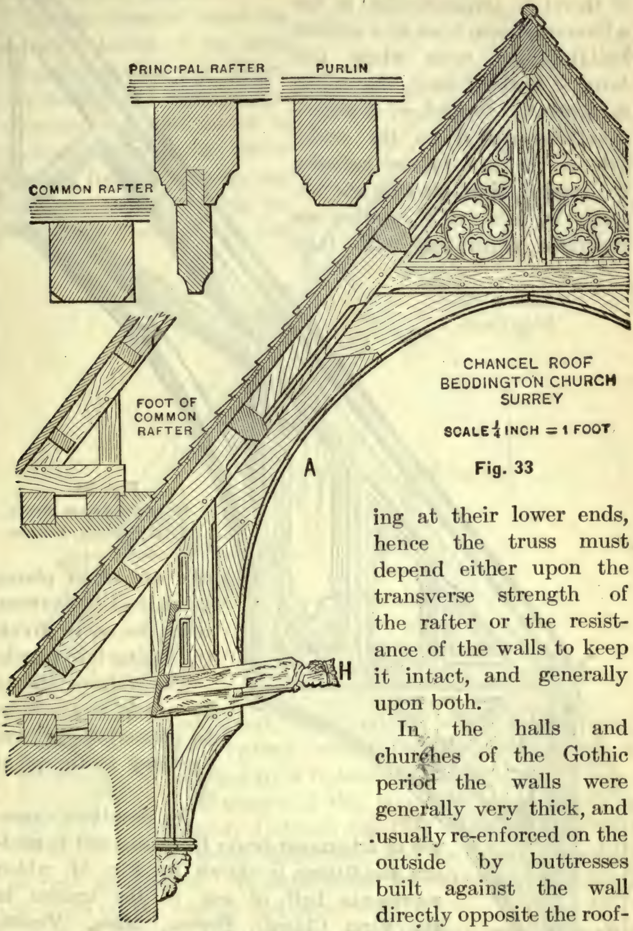


Fig. 32
ROOF OVER NAVE
CHAPEL. SUFFOLK, ENG.
Span 18 ft.

cipals are connected near the top of the truss by a short collar beam, this would offer but little resistance to the rafters spread-



CHANCEL ROOF
BEDDINGTON CHURCH
SURREY

SCALE $\frac{1}{4}$ INCH = 1 FOOT

Fig. 33

ing at their lower ends, hence the truss must depend either upon the transverse strength of the rafter or the resistance of the walls to keep it intact, and generally upon both.

In the halls and churches of the Gothic period the walls were generally very thick, and usually re-enforced on the outside by buttresses built against the wall directly opposite the roof-trusses. In most cases such a wall possesses sufficient stability to withstand the thrust of the truss, and hence the tie-beam may be dispensed with; but in a wooden building the walls offer no resistance whatever to being thrust out, unless tied at the top, and hence no truss which exerts an out-

ward thrust on the walls should be used in such a building. It is therefore impracticable to use a hammer-beam truss in a wooden building. In roofs where this form of truss is used, the ceiling is generally formed of matched sheathing nailed to the under side of the jack-rafters between the purlins, thus allowing the latter to be seen. The purlins are generally decorated, and false

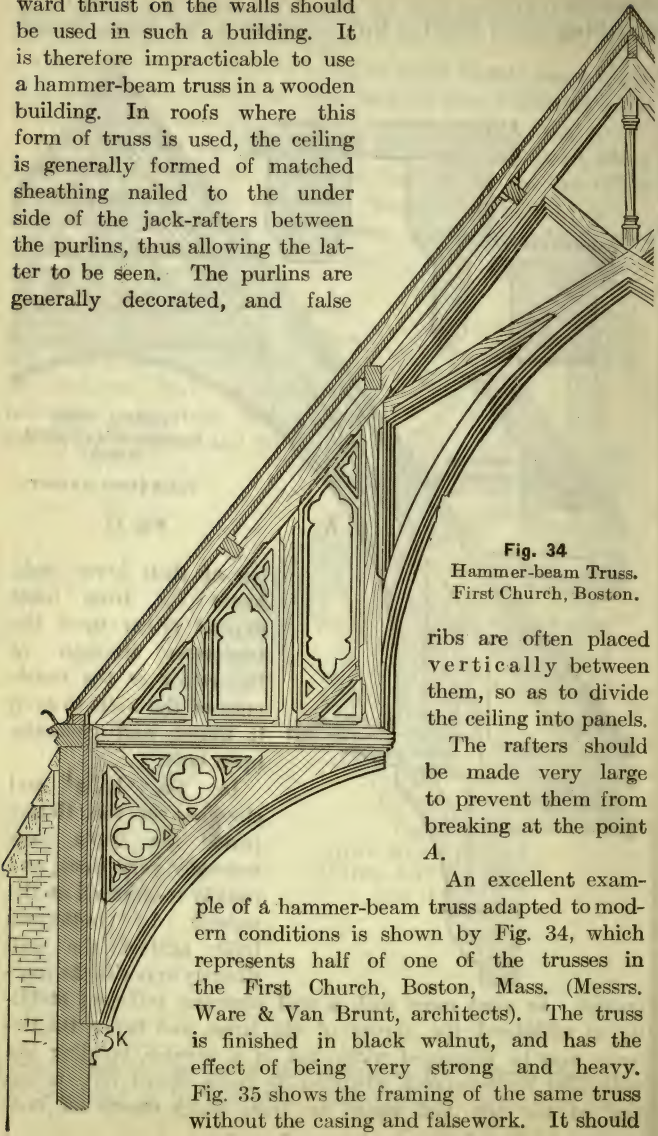


Fig. 34
Hammer-beam Truss.
First Church, Boston.

ribs are often placed vertically between them, so as to divide the ceiling into panels.

The rafters should be made very large to prevent them from breaking at the point *A*.

An excellent example of a hammer-beam truss adapted to modern conditions is shown by Fig. 34, which represents half of one of the trusses in the First Church, Boston, Mass. (Messrs. Ware & Van Brunt, architects). The truss is finished in black walnut, and has the effect of being very strong and heavy. Fig. 35 shows the framing of the same truss without the casing and falsework. It should

be noticed that inside the turned column at the upper part of the truss (Fig. 34) there is an iron rod (Fig. 35) which holds up the joint A.*

In this form of truss the outward thrust of the arch enters the wall just above the corbel, K; and, as the direction of the thrust is inclined only about thirty degrees from a vertical, the tendency which it has to overthrow the wall is not very great, and may be easily resisted by a wall twenty inches or two feet thick, re-enforced by a buttress on the outside.

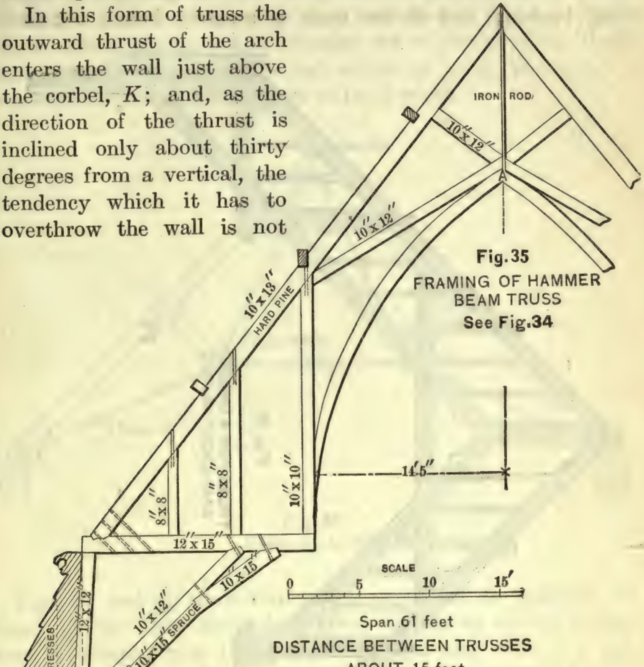


Fig. 35
FRAMING OF HAMMER BEAM TRUSS
See Fig. 34

Span 61 feet
DISTANCE BETWEEN TRUSSES ABOUT 15 feet

In trusses of this kind, the pieces should be securely fastened together wherever they cross or touch each other, and the whole truss made as rigid as possible. No dependence for extra strength should be made on the casings and panel work.

Fig. 36 shows a form of truss used in Emmanuel Church at Shelburne Falls, Mass. (Messrs. Van Brunt & Howe, architects).

* The main rafters of this truss are two five-inch by thirteen-inch hard-pine timbers.

This truss was probably derived from the hammer-beam truss, and possesses an advantage over that truss in that it has in effect a trussed rafter, so that there is no danger of the rafter being broken; and if the truss is securely bolted together

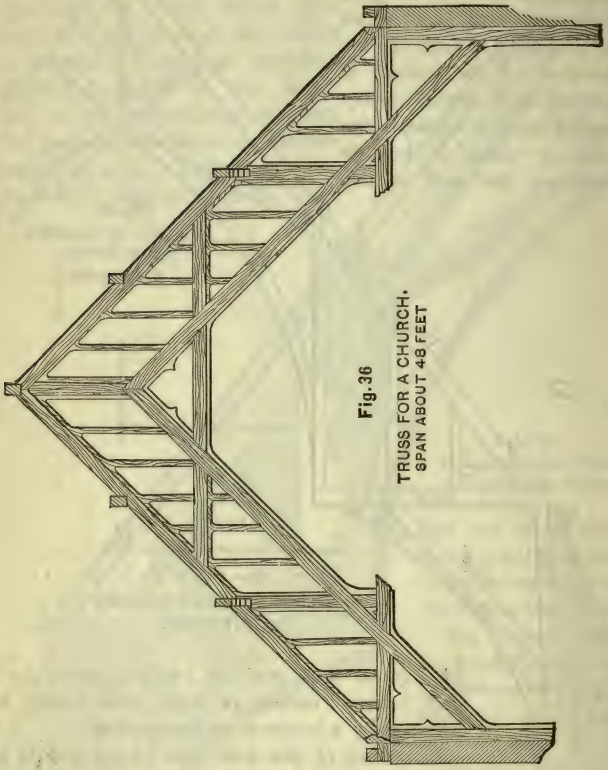


Fig. 36
TRUSS FOR A CHURCH.
SPAN ABOUT 48 FEET

at all its joints it exerts but very little thrust on the walls. The rafters and cross-tie are formed of two pieces of timber bolted together, and the small upright pieces run in between them.

The trusses in the church at Shelburne Falls have the hammer-beams carved to represent angels.

The action of the stresses in hammer-beam trusses is explained in Chapter XXVI.

Wooden Trusses with Iron Ties.—Where there is no ceiling beneath the roof, and it is desirable to make the trusses as light in appearance as possible, wrought-iron or steel rods may be used for the ties, still retaining the wooden principals and struts. Such trusses will be cheaper, for moderate spans, than steel trusses, while they are just about as good, particularly where the rafters and purlins are to be of wood.

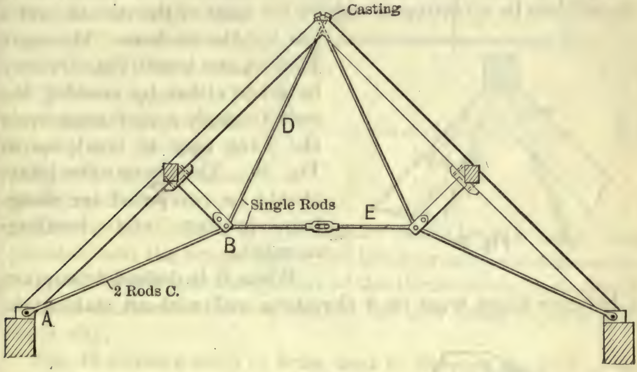


Fig. 37
Spans up to 36′.

Figs. 37 and 40 show forms of trusses that are suitable for many places. The dimensions given in Fig. 40 are for yellow pine or Oregon pine timber and wrought-iron rods, and are ample for a slate roof, the trusses to be spaced from 12 to 14 feet on centres. Trusses like Fig. 37 are sometimes seen with the rods *C* and *D* continuous. They should not be made in this way, however, as the stress in *C* is greater than that in *D*. The best way of making the connection at joint *B* is shown by Fig. 38, a cast-iron shoe being fitted to the end of the strut to receive the pin. For the truss shown by Fig. 40, a shoe made as shown in the detail drawing will make a better connection for the rods, two of the rods

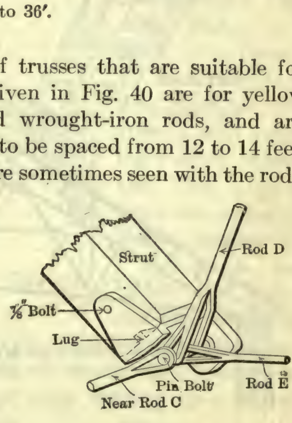


Fig. 38
Detail of joint B.

being placed outside of the brackets, and three between the brackets.

For a truss with a single strut, a turnbuckle on the rod *E* will be sufficient to tighten the rods. When there are three struts, there should be five turnbuckles, as in Fig. 40.

A cast-iron shoe should be made to receive the foot of the rafter, and the rods secured to a pin passed through the shoe and the rafter. At the apex of the truss shown by Fig. 40 there should also be a casting to receive the ends of the rafters, and a pin for the tie-bars. The apex joint of the truss (Fig. 37) may be made either by crossing the rods through a cast washer, or the joint may be made as in Fig. 39. The pins at the joints should be computed for shearing, bearing and bending-moment.

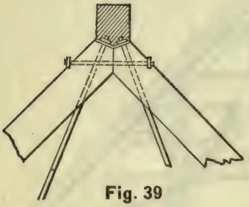


Fig. 39

When it is desired to support a hammer-beam truss on a clerestory wall without making the

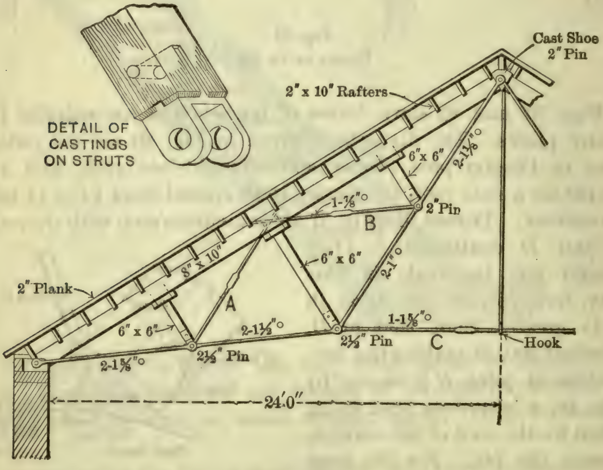


Fig. 40

wall very thick or bracing it from the outside, a form of truss

like that shown in Fig. 41 may be used to advantage. This truss has the appearance of a hammer-beam truss, and when placed over a high nave the effect of the rods is not objectionable.

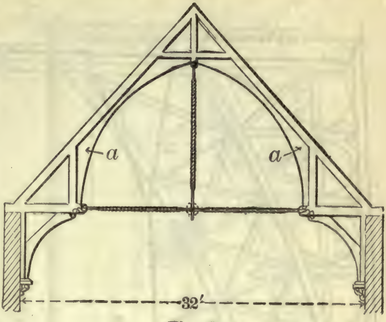


Fig. 41

The tie-rods should extend through the hammer beams to their outer end. For a truss of 32-feet span a $1\frac{1}{4}$-inch square bar will be ample, and it may be twisted to give a more pleasing effect.

The curved ribs *a*, *a*, in this truss are not in tension but in compression, and the braces under the hammer-beams are necessary to resist the vertical component of the thrust in the curved ribs. A truss similar to this was used in the new Grace Chapel, New York city.

Fig. 42 shows a form of truss used to support the roof of the Metropolitan Concert Hall, New York city, George B. Post, architect. The span of the truss in that building is about 54 feet, and the proportions are about as shown in Fig. 42.

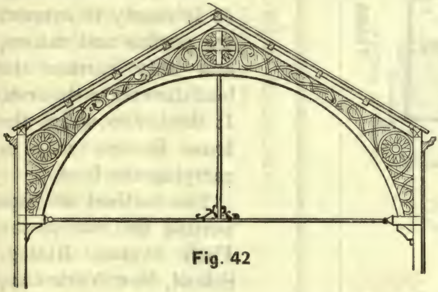
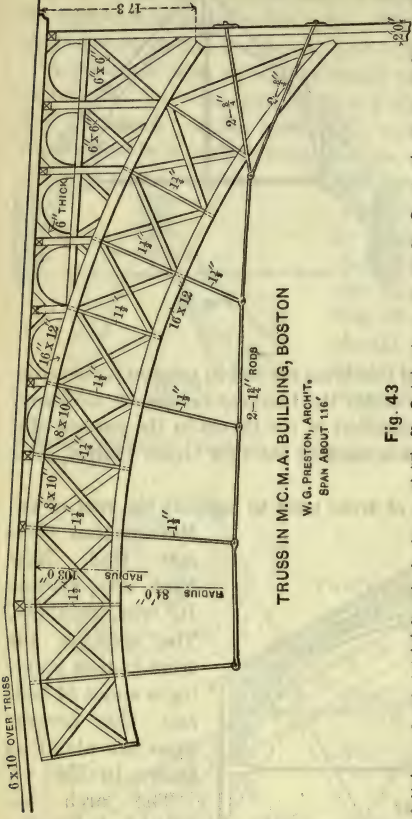


Fig. 42

The arch between the rafter and the raised rib is ornamented with sawed work. The truss has a very light and airy appearance, besides embodying all the strength that can be desired in it. The tie-rod is kept from sagging by a vertical rod from the centre of the arch.

Wooden Arched Ribs, with Iron or Steel Ties.—For roofing large halls or rooms a segmental timber arch, with an iron or steel tie for taking up the horizontal thrust makes about

the cheapest truss that can be built, especially where there is no ceiling to be supported.



TRUSS IN M.C.M.A. BUILDING, BOSTON
W. G. PRESTON, ARCH'T.
SPAN ABOUT 116'

Fig. 43

Figs. 43 and 44 are good examples of this form of truss. The arched ribs support all the load that comes upon the truss, and the tie-rods prevent the ends of the arch from spreading, as would be the case if there were no tie-rods.

The bracing between the arched ribs is simply to unite them, and distribute the stresses arising from the load proportionately over the two ribs.

The framework shown above the arch in Fig. 43 is simply to support the purlins and rafters, and only carries the load directly to the arch. It does not assist the truss in any way in carrying the load.

The method of supporting the roof of the Fifth Avenue Riding-School, New York city, is slightly peculiar and very ingenious; and, as it is an excellent example of the advantage of the arched form of truss, we shall give a brief description of the construction of the roof and its supports. A plan of the riding-room is represented by Fig. 45. The room is one hundred and six feet six inches long, and seventy-three feet wide. This space is kept entirely clear of posts or columns; and the entire roof is supported by two large trusses, one of which is shown in Fig. 44. The roof be-

tween the trusses and on either side is supported by smaller trusses resting on these large trusses; but each of the large trusses eventually carries a roof area equal to about 2,930 square feet, and a great amount of extra framework. It was desired to provide for the thrust of these large arches without having rods showing in the room, and the method adopted is very ingenious. Opposite the upper ends of the iron posts which receive the arched ribs are oak struts, which are held in place by iron tie-bars and heavy iron beams, which together form a horizontal truss at each end. These two trusses are prevented from being pushed out by two three-inch by one-inch tie-bars in each side-wall shown in the plan (Fig. 45).

The bottoms of the two iron posts are tied together by iron rods running under the floor the whole length of the room. Altogether this gives for the tie-rods of each truss two bars three inches by one inch, and an inch and a half iron rod, which would be equivalent to two tie-bars three inches and

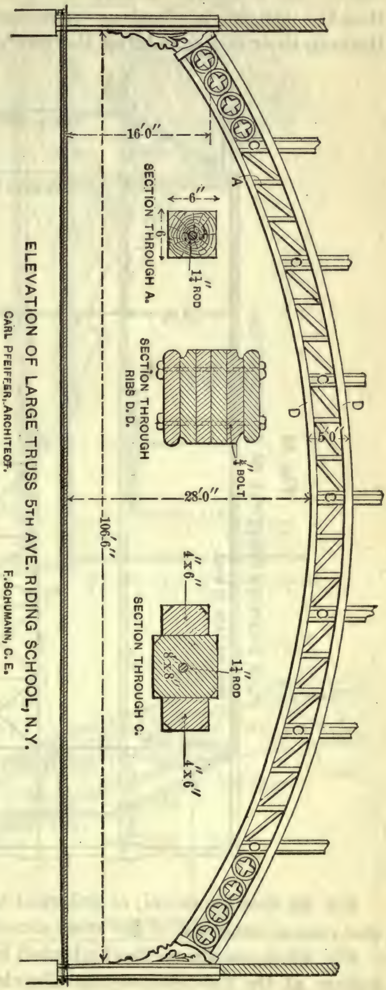


Fig. 44

ELEVATION OF LARGE TRUSS 5TH AVE. RIDING SCHOOL, N.Y.

CARL PFEIFFER, ARCHITECT. F. SCHUMANN, C.E.

three-fourths by one inch. Enlarged sections of the ribs, uprights, and braces are shown in Fig. 44. It should be noticed that the uprights act both as struts and ties, by having iron rods through their centre holding the two ribs together.

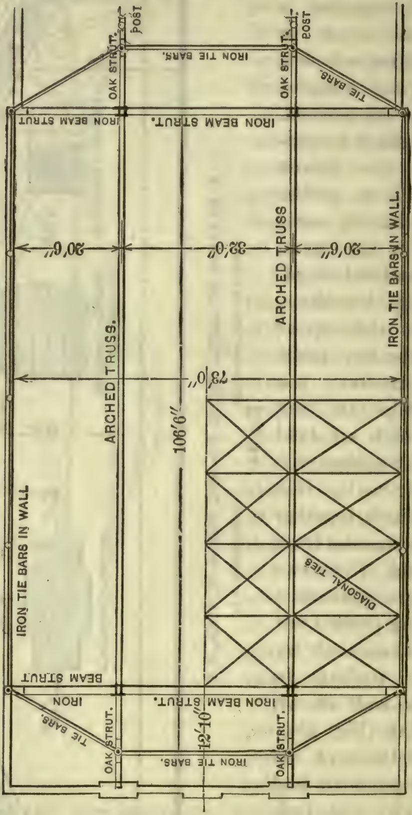


Fig. 45

PLAN SHOWING PRINCIPLE OF CONSTRUCTION.

Fig. 46 shows a detail, or enlarged view, of the iron skewback and post at each end of the truss shown in Fig. 44.

Fig. 47 shows the method adopted for supporting the roof and gallery at the City Armory at Cleveland, O., the arch being of wood.

Fig. 48 shows one-half of an arched wooden truss which, with seventeen others, was designed for supporting the roof over the

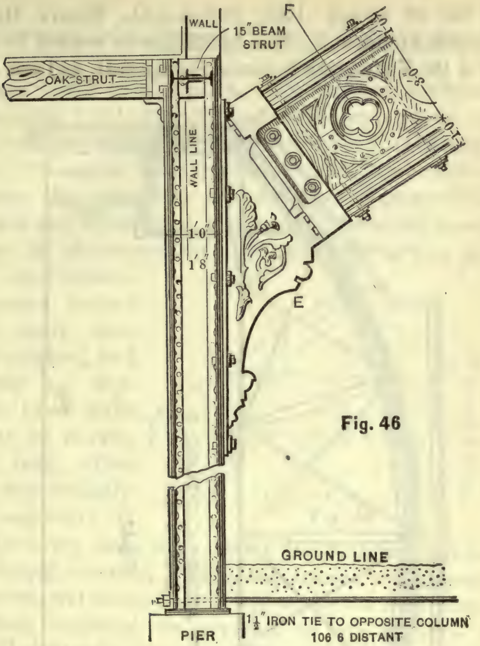

F
WALL
15" BEAM STRUT
OAK STRUT
WALL LINE
1'-0"
1' 8"
3'-0"
E
GROUND LINE
PIER
1½" IRON TIE TO OPPOSITE COLUMN 106 6 DISTANT

Fig. 46

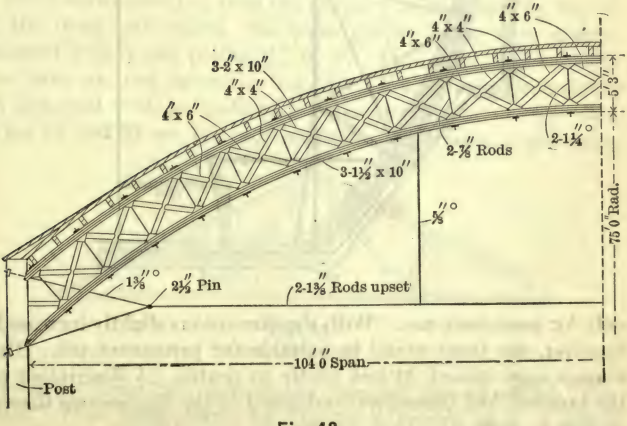

4" x 6"
4" x 4"
4" x 6"
3-2" x 10"
4" x 4"
4" x 6"
5' 3"
2-⅞ Rods
2-1¼"○
3-1½" x 10"
⅝"○
75' 0" Rad.
1⅜"○
2½" Pin
2-1⅜" Rods upset
104' 0" Span
Post

Fig. 48

central bay of Saenger Hall, Philadelphia, Messrs. Hazelhurst and Huckel, architects. This building was erected in 1897 for the use of the Eighteenth National Saengerfest, and was intended

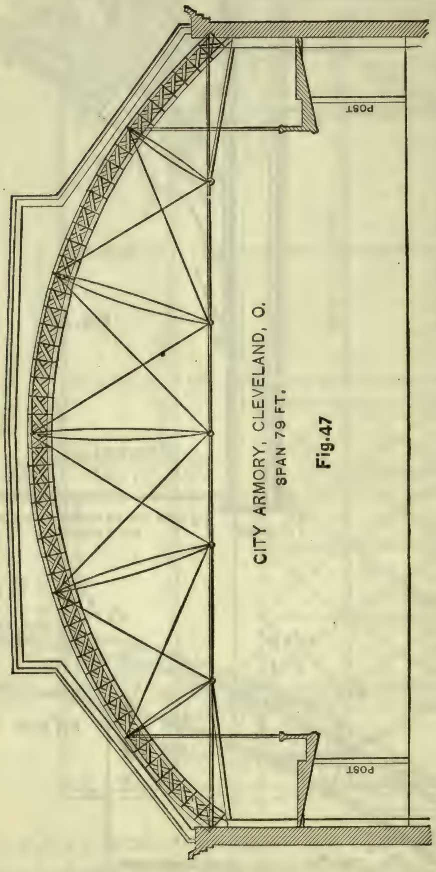


Fig.47

only for temporary use. With the dimensions slightly increased, however, the truss would be suitable for permanent use. The trusses were spaced 20 feet centre to centre. A description of the building and trusses was published in the *Engineering Record* of Jan. 9, 1897.

TYPES OF STEEL ROOF TRUSSES.

Trusses for Pitch Roofs.—For ordinary conditions and for spans under 100 feet, some one of the types shown by Figs. 49 to 60 will generally meet the requirements of strength and economy. Trusses of these types are usually made with riveted connections, this being the cheaper kind of construction for short spans and small truss members. There are cases, however, when the pin connection may be the cheaper or more advisable construction. Pin-connected trusses may be more conveniently shipped, and where they are supported by brick walls so as not to require bracing, may often be more economically erected, especially if there is no other steel work about the building that requires riveting during erection. When the trusses are supported by steel columns, and where there is a good deal of steel work about the building requiring the presence of iron-workers, riveted trusses will always be more economical for spans up to 80 feet.

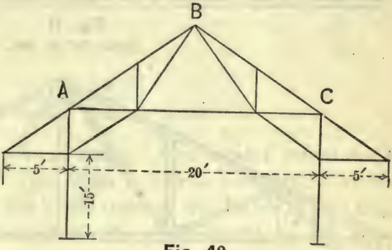


Fig. 49

For a narrow shed or shop the shape of truss shown by Fig. 49 is the most economical, the truss proper being that portion inclosed within the points *A*, *B*, *C*. This truss is practically the same as that shown by Fig. 50. For spans of from 24 to 48 feet, and with an inclination not exceeding 6″ to the foot, types 51 and 52 are the most suitable. The truss types represented by these two figures has received the name of "Fan truss." The truss shown by Fig. 50 is known as a "simple

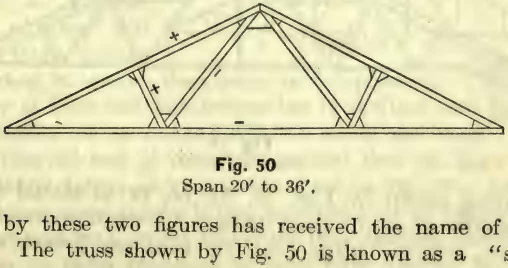


Fig. 50
Span 20′ to 36′.

Fink truss." The truss shown by Fig. 52 differs from that in 51 principally in the inclination of the braces, the braces *A* and *B* in Fig. 52 being inserted to brace the truss from the column to prevent racking under wind pressure. Fig. 52

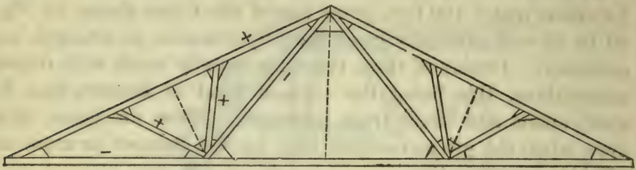


Fig. 51
Span 36′ to 50′.

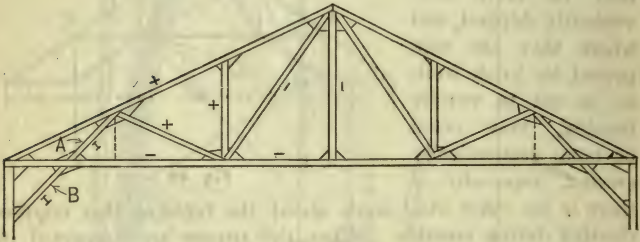


Fig. 52
Span 40′ to 60′.

should be used when the truss is supported by columns and Fig. 51 when the truss rests on brick walls. A sag tie, as shown by dotted line, is generally inserted. When the roof construction demands three purlins on each side of the truss, one of

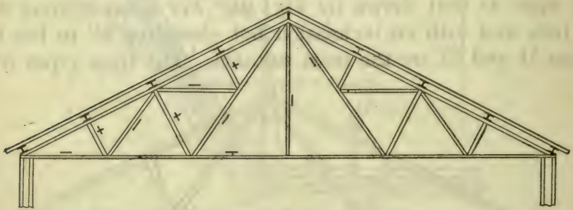


Fig. 53
Span 40′ to 80′.

the forms shown by Figs. 53, 54, 55, or 56 should be used. The names given to these trusses are often confounded by different writers; many engineers class the French and Fan

trusses with the Fink truss. The term "French" appears to be generally given to those trusses in which the tie-beam is raised in the centre. The truss shown by Fig. 56 appears to

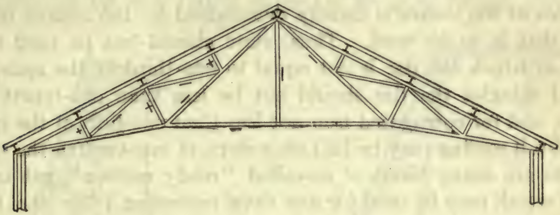


Fig. 54

French Truss. Span 40′ to 80′.

have no generally recognized name. One writer refers to it as an "English" truss. This truss is not as economical as the Fink truss, except when the inclination of the rafter is less than one-fourth pitch, on account of the great length of the inner struts.

Although Fig. 56 somewhat resembles the queen truss, Fig. 5, it will be seen that the diagonals run in the opposite direction, the diagonals in Fig. 56 being in tension, and the verticals in compression, the reverse of the queen truss. In designing steel trusses it is desirable to have as many members, and especially of the long members, in tension as possible, as a given weight of steel will resist a much greater stress when in tension than when in compression. The great economy of Fink and Fan trusses lies in the fact that most of the members are in tension and the struts are short. Comparing Figs. 55 and 56 it will be noticed that the inner strut in Fig. 55 is only $\frac{1}{3}$ as long as the strut in Fig. 56. Another advantage of these trusses is that a partial load, as, for instance, a wind or snow load, on one side of the truss never causes stresses in excess of those produced by a uniform load of the same

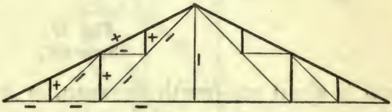


Fig. 55

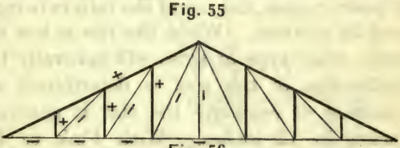


Fig. 56

intensity over the whole truss. If the roof is hipped it is desirable to have vertical members in the hip trusses to receive the short trusses or trussed purlins.

Depth of Fink and Fan Trusses.—The depth of these trusses at the centre is usually determined by the roofing material that is to be used. Thus, slate should not be used on a roof in which the rise is not equal to one-third of the span, for wood shingles the rise should not be less than one-fourth the span, and for corrugated iron not less than one-fifth of the span. Steel-roll roofing may be laid on a slope of one-twelfth the span. There are many kinds of so-called "ready roofing" put up in rolls which may be used for any slope exceeding $\frac{3}{4}''$ to the foot. Tar and gravel roofing should never be used on a pitch exceeding $\frac{5}{8}''$ to the foot. Considering the construction of the roof and the weight of the trusses the most economical pitch for a

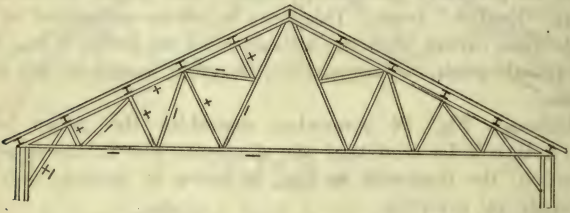


Fig. 57
Span 68′.

roof is about one-fourth the span, or what is commonly called a quarter pitch, the rise of the rafters being 6″ in 12″, or 26 degrees and 34 minutes. When the rise is less than one-sixth the span some other type of truss will generally be required. When the inclination of the roof is determined almost entirely by the question of economy the rise is generally made from 6 to 7 inches in 12 inches. With Fink or Fan trusses having an inclination for the rafter not exceeding 30 degrees it is more economical to employ a horizontal chord or tie, since it obviates bending of the laterals. A truss whose bottom chords has a rise of two or three feet, as in Fig. 54, presents a better appearance, however, than one with a horizontal chord. Raising the bottom chord also materially increases the strains in the truss members, hence it increases the cost. For steep roofs, however, it will generally be fully as economical to raise the bottom chord, because of the shortening of the members.

Number of Struts.—The number of struts that should be used in each half of the truss will be determined in a great measure by the construction of the roof. If Jack rafters and purlins are used then the distance between the struts may be as great as 12 feet, but if there are no Jack rafters and the

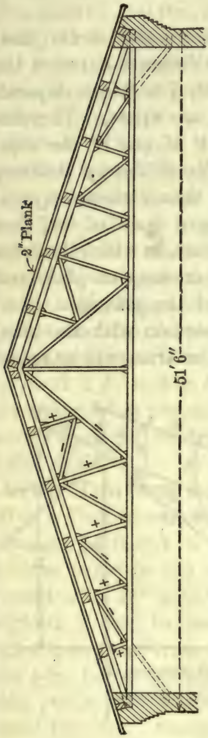


Fig. 53

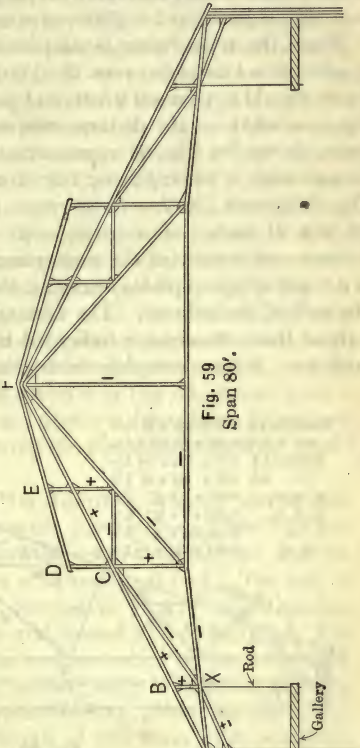


Fig. 59
Span 80'.

planking of the roof is nailed directly to the purlins, then the latter will not be placed more than 8 feet apart, and if the roof is covered with corrugated iron secured to the purlins, then the purlins should not be more than 5 feet on centres. Whenever the purlins are more than four feet apart they should come over the end of a strut or brace, to avoid bending-moments, consequently the spacing of the purlins will generally deter-

mine the number of struts in each half of the truss. For this reason the same form of truss may be required for a span of 40 feet as for a span of 80 feet, but of course the members will not be as heavy in the 40-foot truss as in the one with greater span. The trusses shown by Figs. 50 to 60 are mostly drawn from actual cases, and give a pretty good idea of the most economical division for different spans.

When the truss rafter is subject to a transverse strain, that is when it is loaded between the joints, the distance between the joints should not exceed 9 feet, and preferably 7 or 8 feet, depending somewhat on the distance the trusses are apart. The diagram shown by Fig. 60 represents one-half of one of the steel trusses used in roofing a car barn for the North Jersey Railway Co. at Newark, N. J. There were 13 of these trusses spaced 19 feet $2\frac{1}{4}$ inches on centres, each having a span of $98\frac{1}{4}$ feet between the centres of the supporting columns, to which the truss is riveted by splice-plates engaging the end connection-plate and the web of the column. The dimensions of the principal members of these trusses are indicated in connection with the illustrations. A more complete description of the truss will be found

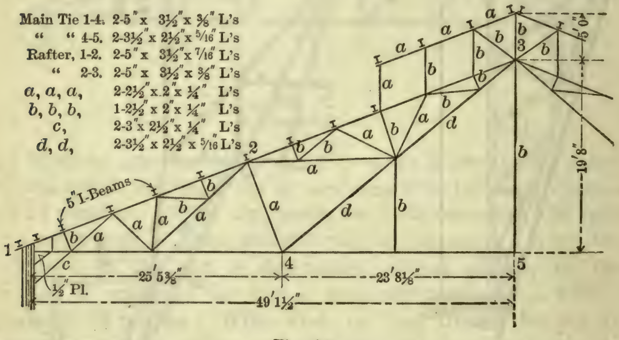


Fig. 60
Truss over Car Barn, Newark, N. J.

in the *Engineering Record* of June 22, 1901. These trusses were shipped in four sections, which were assembled in a horizontal plane and riveted up complete at the surface of the ground. The bottom chord was stiffened by lashing a rail on each side of it for its entire length, and a sling being attached to the apex of the top chord the truss was lifted and set on top of the columns

by an 8×8 gin-pole 50 feet high. The roofing consists of corrugated iron supported by 5-inch I-beam purlins, weighing 10 lbs. to the foot, spanning from truss to truss and bolted to the rafters with two bolts at each end; the general spacing of the purlins being 4 feet 9¾ inches. This may be considered as an example of an extremely light roof; the weight of each truss being about 4,200 lbs., and the entire weight of the truss, purlins, bracing of the lower chord and the corrugated iron roofing being only 8 lbs. for each horizontal foot of surface covered. The trusses shown by Fig. 59 were designed for the roof of a drill-hall having a span of 80 feet, and with a spacing between the trusses of 20 feet. The roof was to be constructed with 2″×8″ rafters supported by purlins at points *A, B, C, D, E,* and *F*. Sash were to be placed in the rise *C D*, to light the interior of the building. The joint at *X* was located with reference to the position of the gallery rod; if there had been no gallery it would have been more economical to space the vertical struts uniformly as in Fig. 55. The plus sign adjacent to a member in all the trusses illustrated denotes that the member is in compression, while the minus sign denotes tension. The members above the main rafter, as *C D, D E,* and *E F*, in Fig. 59, and *a* and *b* in Fig. 60, do not form a part of the truss proper, but are merely a framework to support the elevated roof, and in drawing the stress diagram they would be omitted.

Fink Trusses with Pin Joints.—Fig. 61 shows one-half of a Fink truss designed for pin connections. This truss has a span of 55 feet 4 in. between the centres of end-pins, and the distance between the centres of trusses is 6 feet. The roof is covered with 12″×20″ slate, secured to ½″×2¼″ angle purlins weighing 3 lbs. to the foot and spaced 8½″ on centres. The angles span from truss to truss and are bolted to the deck-beam with ½-in. bolts. A 1½″×2¼″ nailing strip is fastened to every third purlin for securing matched ceiling placed on the under side of the roof. Complete details of this truss were published in *Architecture and Building* for Jan. 18, 1890. Fig. 62 shows details of the cast-iron struts. This truss, being put together entirely with bolts and pins, could easily be erected with unskilled labor.

Trusses for Flat Roofs.—For supporting flat roofs or roofs having a fall not exceeding 1 inch to the foot, one of the types shown by Figs. 63 to 67 will generally be found economical, the choice of the particular type depending

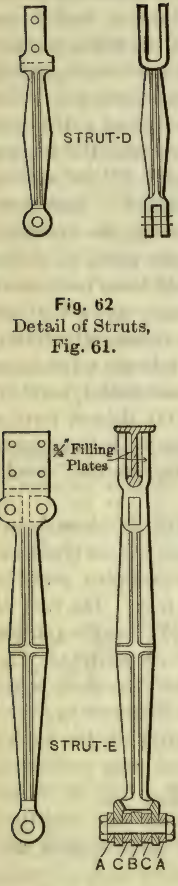


Fig. 62
Detail of Struts,
Fig. 61.

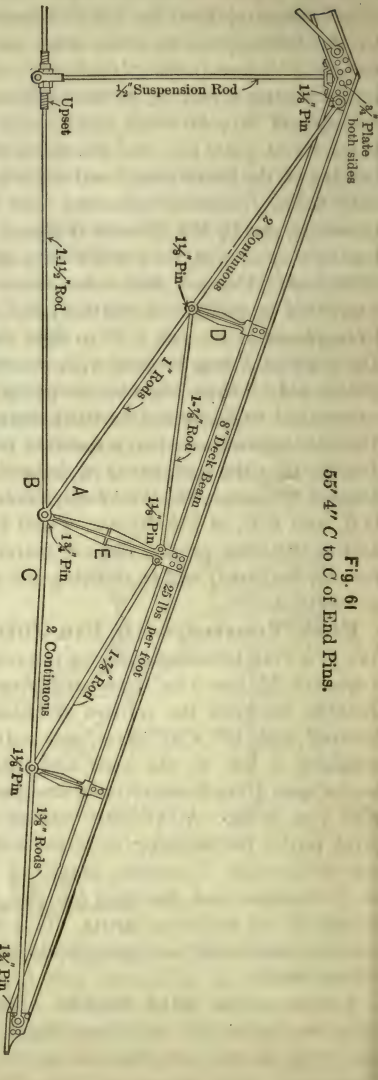


Fig. 61
55′ 4″ C to C of End Pins.

somewhat on the span and whether the truss is supported by columns or by brick or stone walls. For spans up to 50 feet either of the forms shown by Figs. 63 or 64 will answer

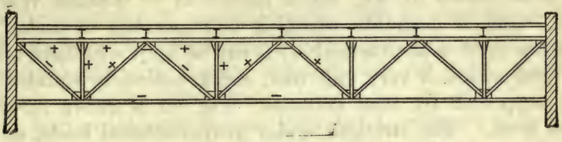


Fig. 63
Span 56′.

all practical requirements. The truss shown by Fig. 63 is intended to be used where the fall of the roof is at right angles to the truss; this truss can be built, however, with an inclination to the top chord, as in Fig. 64. The end brace in Fig. 63 is in

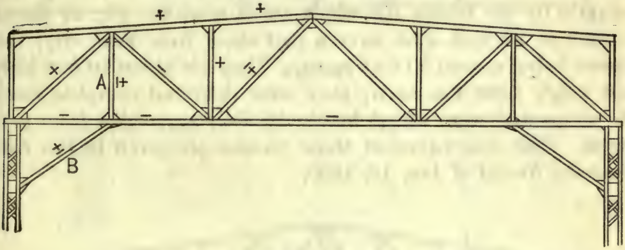


Fig. 64
Spans 30′–50′.

tension, while in Fig. 64 it is in compression. The portion of the lower chord between the end joint and the wall, Fig. 63, has

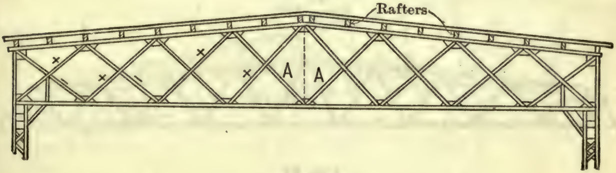


Fig. 65

no stress from the roof load, but is put in to brace the wall and to stay the truss. In trusses supported by brick walls this type is preferable to that shown by Fig. 64, while the latter is

more suitable when the roof is supported by columns. The vertical A, Fig. 64, is inserted to receive the tension or compression from brace B, and would have no stress from the roof. The truss shown by Fig. 65 is known as a "Double Warren Truss," and is desirable where it is important to make the trusses as shallow as practicable; it can be built with light members, and makes a very stiff roof, being especially suitable for roofs supported by steel columns. Fig. 65 is drawn from an actual truss. The strength under unsymmetrical loads, as for example when there is more snow on one side than on the other, would be materially increased by putting a vertical tie in the centre as shown by the dotted line; without this member the braces AA if subject to any stress whatever would produce a bending in the bottom chord at the centre. Fig. 66 represents an actual roof truss with span of 57 feet, supported by steel columns. The entire load on the truss is transmitted to the columns by the braces BB which are in tension. Fig. 67 shows a truss of 96 feet span over a pier shed, New York city, the trusses being spaced 20 feet apart. They are about 10 feet high and weigh 1,300 lbs. each; they were delivered complete from the shops and were raised bodily by falls suspended from two masts. The dimensions of these trusses are given in the *Engineering Record* of Jan. 18, 1896.

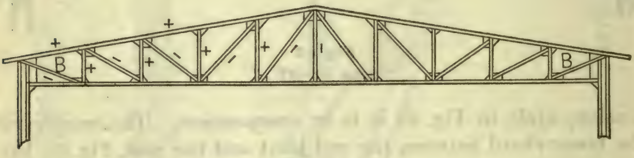


Fig. 66
Span 57′.

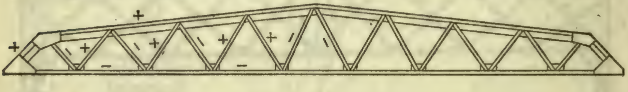


Fig. 67
Span 96′–′.0

The plus and minus signs in these illustrations indicate compression and tension respectively under uniform dead load. The plus and minus signs together indicate that the member may

be subject to either tension or compression according to the direction of the wind or to an uneven distribution of snow. In most of these trusses an unsymmetrical load may change the stress in the diagonals near the centre of the truss. This changing of stresses due to unequal loading will be considered in the next chapter. Trusses shown by Figs. 63 to 67 are almost invariably built with riveted connections and with angle or channel shapes for all members.

For horizontal steel trusses intended to support floor loads, the Pratt truss, shown by Figs. 68 and 69, is the best adapted, the members indicated by double lines being in compression and those indicated by single lines in tension. When supporting

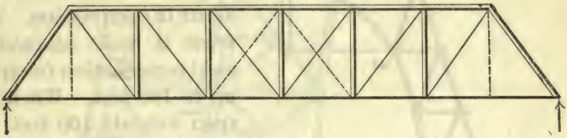


Fig. 68

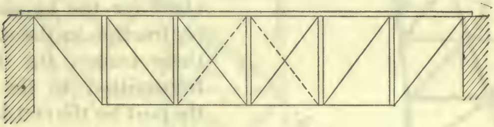


Fig. 69

floors subject to moving loads, counter ties should be inserted as indicated by dotted lines. For this truss, pin-connections are generally employed and are preferable to riveted connec-

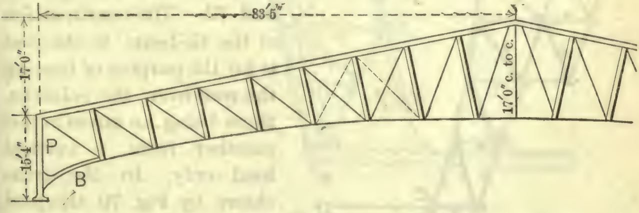


Fig 70.

Truss over Amphitheatre, Madison Square Garden.

tions. When properly proportioned this truss is capable of sustaining almost any load.

The Quadrangular Truss.—The truss shown by Fig. 66 is known as a quadrangular truss, although the more common shape for this truss is that shown by Fig. 70, which gives the proportions of the truss over the amphitheatre of the Madison Square Garden, New York. Figs. 71 and 73 also show variations of this truss, differing, however, from the typical truss, in that the diagonals are all inclined in the same direction, while in the typical truss they are usually reversed in the centre in order to keep them in tension.

The plus and minus signs indicate the kind of stress in the member produced by a uniform dead load. It should be noticed that the centre diagonals of trusses 71 and 73 are in compression. This truss is well adapted to steel construction for spans up to 180 feet. When the span exceeds 100 feet one end of the truss should be supported on rollers to allow for the expansion or contraction in the steel. In these trusses the load is transmitted to the top of the post by the end diagonal which is always in tension and subject to a very great stress, the truss proper being included within the points *A*, *B*, *C*, *D*, and *E*, Figs. 71 and 72. The continuation of the tie-beam to the post is for the purpose of bracing the roof from the columns, there being no stress in this member from a vertical load only. In the truss shown by Fig. 70 the post *P* was made a part of the truss; the stress in this post is equal to the reaction of the truss. The brace *B*, Fig. 70, and the corresponding member

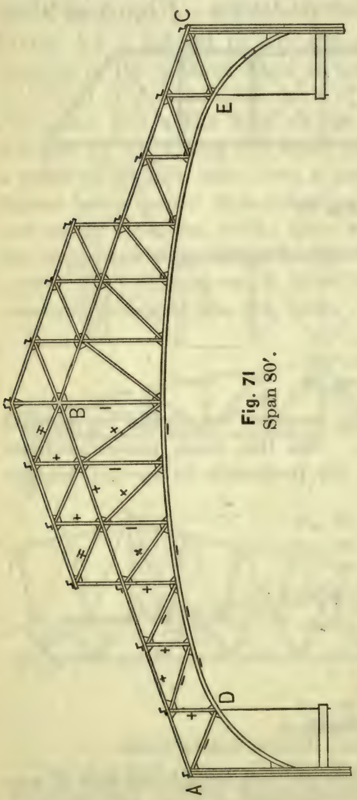


Fig. 71
Span 80′.

in Figs. 71 and 72 should be so constructed as to resist both tension and compression. For short spans the lower chord

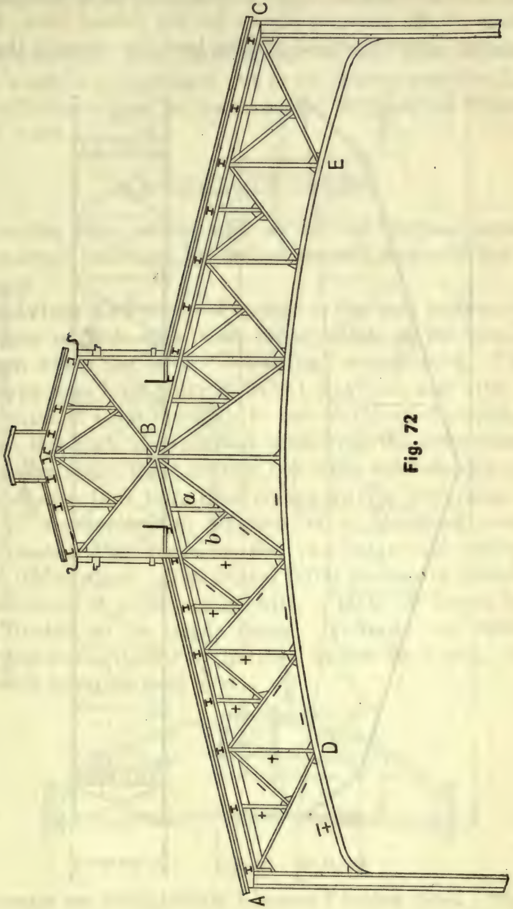


Fig. 72

may be made in the shape of a semi-circle or semi-ellipse so as to give more of an arch effect.

There are numerous examples in this country of quadrangular trusses having spans of from 100 to 180 feet. For the wider

spans it is customary to build the truss with pin-connections, eye-bars being used for the ties. When this is done it will usually be necessary to insert counter braces in two panels on each side of the truss as shown by the dotted lines, Fig. 70, as under an unsymmetrical or wind load the stress in the diago-

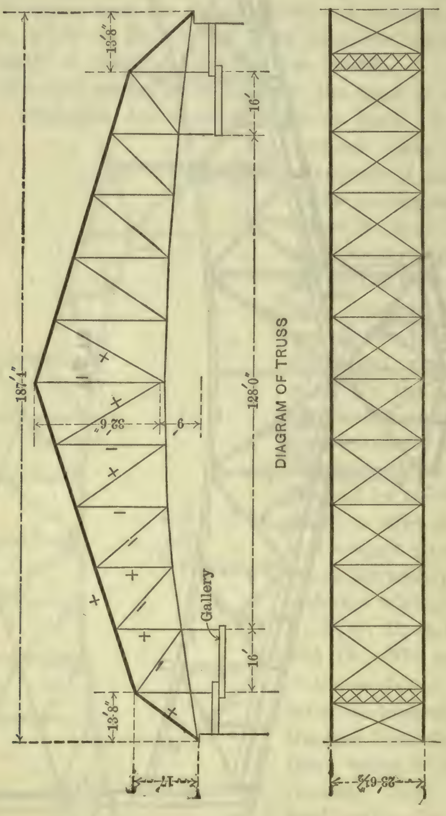


Fig. 73
Plan of two trusses, showing lateral bracing.

nals is generally reversed. When the span is less than 100 feet, the truss may be built with riveted connections, in which case the diagonals are generally made of angles capable of resisting both tension and compression, and, therefore, counter braces will not be required. For this type of truss the stresses

due to wind and snow should be computed independently of the dead load and the members computed for the maximum stress that may be produced by any possible combination of loading.

A description, with illustrations of the truss shown by Fig. 73, which is a diagram of one of the trusses over the Kansas City Auditorium, may be found in the *Engineering Record* for July 22, 1899.

ARCHED TRUSSES.

For roofing large rooms, such as railway stations, armories, and exposition buildings, an arched truss is generally the most economical.

Bowstring Trusses.—Previous to the year 1880 wrought-iron trusses of wide span were mostly built in the form of a bow, from which the term "Bowstring" was derived. Trusses of this type were built from 88 to 211 feet span and with a rise in the centre of from one-fifth to one-fourth of the span. At that time this style of truss was considered the most economical for spans exceeding 120 feet, but since the introduction of the braced arch they have been comparatively little used.

Fig. 74 represents the diagram of a bowstring truss of 153-feet span. The trusses in this particular case are spaced 21 feet 6 inches apart. The arched rafter consists of a wrought-iron deck-beam 9 inches deep, with a plate 10 inches by $1\frac{1}{4}$ inches, riveted to its upper flange. Towards the springing this rib was strengthened by plates 7 inches by $\frac{7}{8}$ inch, riveted to the deck-beam on each side.

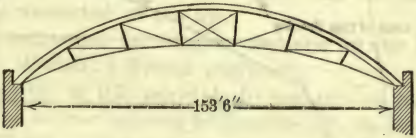


Fig. 74

The struts are wrought-iron I-beams 7 inches deep. The tie-rods have a sectional area of $6\frac{1}{2}$ sq. ins., and the diagonal tension braces are $1\frac{1}{4}$ inches in diameter. These trusses are fixed at one end, and rest on rollers at the other, permitting free expansion and contraction of the iron under the varying heat of the sun.

Fig. 75 shows a similar truss having a span of 212 feet. It consists of bowstring principals spaced 24 feet apart. The

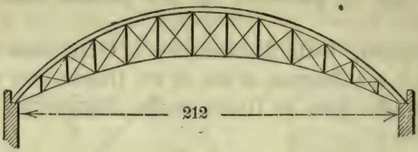


Fig. 75

rise is one-fifth the span, the tie-rod rising 17 feet in the middle above the springing, and the curved rafter rising $40\frac{1}{2}$ feet. The rafter is a 15-inch wrought-iron I-beam. The tie is a round rod in short lengths, 4 inches diameter, thickened at the joints. The tension bars of the bracing are of plate-iron, 5 inches to 3 inches in width, and $\frac{5}{8}$ inch thick. The struts are formed of bars having the form of a cross.

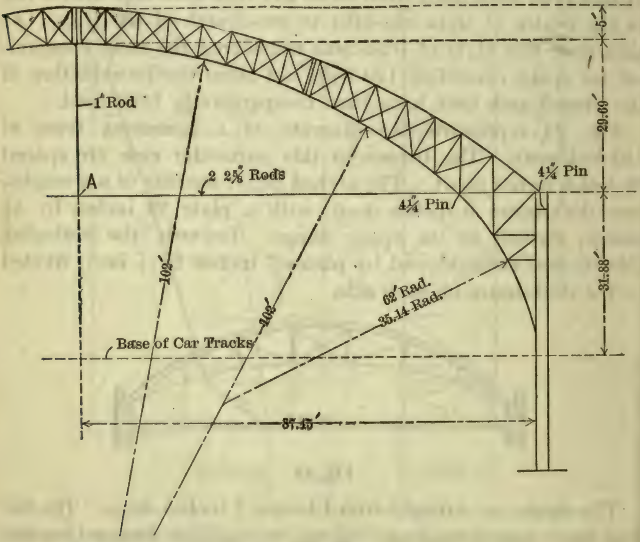
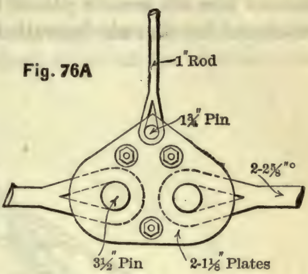


Fig. 76

Braced Arches.—Fig. 76 is a diagram of one of the three arches used in roofing the train shed of the Sullivan Square Station of the Boston Elevated Railway, a description of

which may be found in the *Engineering Record* for June 15, 1901. These arches spring from steel columns and are provided with tension rods which take up the thrust. The arch proper rests on two $4\frac{1}{4}''$ pins at each end as indicated in diagram, the tie-rods being connected to these pins. The bracing below the pins is riveted to the column and the arch itself is built of angles and plates with riveted connections. Fig. 76*A* shows the joint at *A* where the tie-rods are connected and are held up by a 1″ suspension rod from the crown of the arch. This construction is the same in principle as that of the wooden arch shown by Fig. 48. It can hardly be considered as a truss in the ordinary meaning of the word.

Fig. 76A



Three-Hinged, Braced Arches.—The type of arched truss most used at the present time for steel construction is that shown by Figs. 78 and 79, which is commonly known as the three-hinged, braced arch. This truss differs from all other types of trusses in that it consists essentially of two separate parts, each acting as a single piece and depending upon the opposing force of its mate to keep it in position. As usually built, each part is a semi-braced arch, the upper and lower members being so connected by bracing as to form a stiff frame or curved rafter.

The first use of the braced arch appears to have been in building railway bridges for French railroads, the earlier forms being rigidly connected at the top.

The first suggestion for hinging the ribs at the crown was made by M. Manton, a French engineer.

The application of this principle to roof trusses, at least on a large scale, the author believes to have been in the train-sheds of a Union Railway station at Frankfort-on-the-Main, Germany, which was completed in the year 1888. These trusses have a span of about 184 feet. The large roof of Machinery Hall, of the Paris Exposition of 1899, was supported by this type of truss, the span in this case being 368 feet, exceeding anything hitherto attempted in a roof truss. Since then this truss has become quite popular for roofing large exhibition buildings, train-sheds, armories, etc.

The three-hinged arch-truss proper is always supported on a pin at the bottom and usually the two halves are pin-connected at the top, thus allowing for expansion and contraction. The bottom pins are usually placed below the ground floor level and connected by tie-rods beneath the floor. These trusses can be

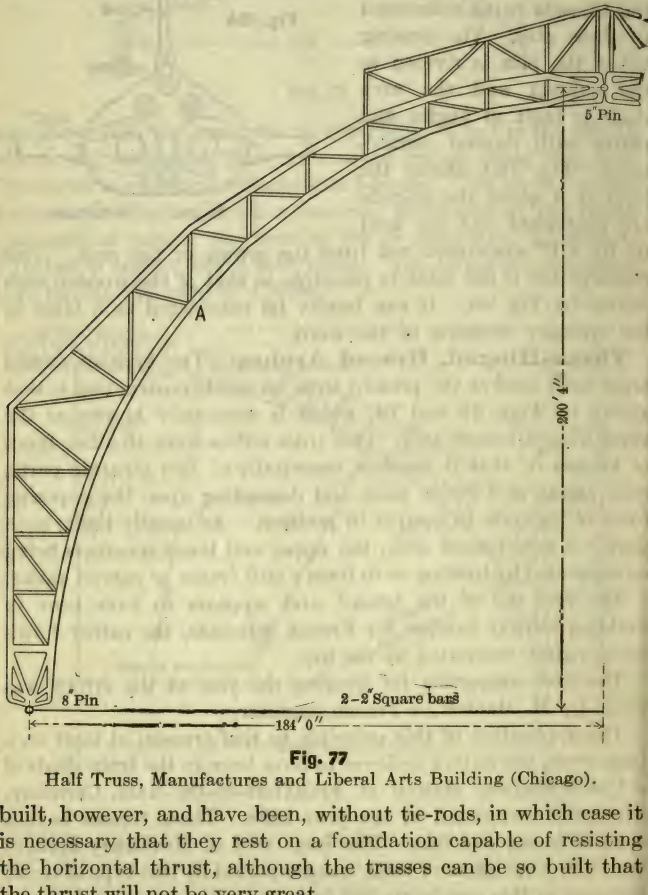


Fig. 77

Half Truss, Manufactures and Liberal Arts Building (Chicago).

built, however, and have been, without tie-rods, in which case it is necessary that they rest on a foundation capable of resisting the horizontal thrust, although the trusses can be so built that the thrust will not be very great.

The special advantages of this type of truss for the class of buildings above mentioned are economy, maximum clear space beneath the truss and provision for expansion and contraction.

Much of the economy of the truss lies in the fact that it requires no columns to support it, and the base of the truss being very near the ground level, it is well-proportioned to resist wind pressure.

A great advantage of this truss is the free movement allowed under temperature changes without strain to the structure, the

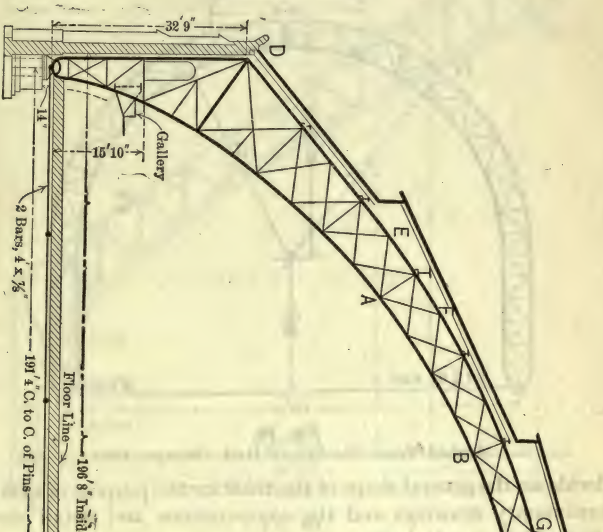


Fig. 78

centre rising or falling freely with a slight rotation of the semi-arches about the pivots. In the case of the trusses of the Paris Exposition, it was estimated that a range of temperature of 100 degrees Fahr. would produce a change in level of $2\frac{1}{8}$ inches at the centre pivot.

The arched ribs are always built of plates, angles or channels

with riveted connections, and frequently with a solid plate web at the bottom.

The determining of the stresses and detailing of the members and joints will require the service of a competent structural engineer, but the illustrations given will enable the architect to

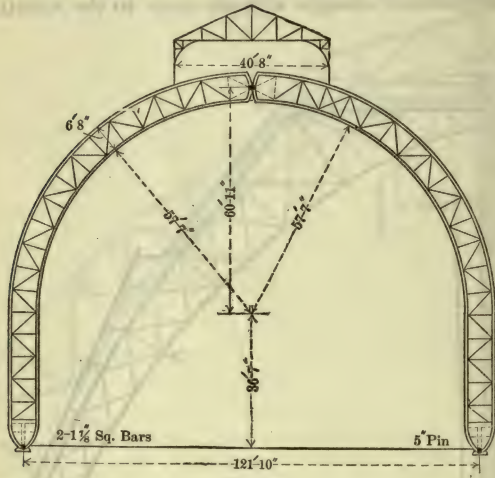


Fig. 79
Arched Truss, Machinery Hall, Chicago, 1890.

decide on the general shape of the truss for the purpose of making preliminary drawings and the computations and detail drawings can be made later.

For spans of from 80 to 120 feet this type is often put up without the pin connections, as in Fig. 80, the mechanical principle being essentially the same as in the three-hinged truss. Complete drawings and details of the truss shown by Fig. 77 may be found in Vol. 26 of the *Engineering Record*, and of the truss shown by Fig. 79 in Vol. 27. A description and details of the truss shown by Fig. 78 may be found in the *Engineering Record* for Dec. 3, 1899. Table VI, Chapter XXVI, gives the dimensions and spacing of a number of trusses similar to Figs. 77, 78, and 80.

Other examples of three-hinged arched trusses are given in Part III of the author's work on Building Construction.

Cantilever Trusses.—The term "cantilever" was originally used to designate a projecting beam which served as a

bracket; in mechanics it is used to denote a beam or girder fixed at one end, either by being built into a wall or, most commonly, by extending a sufficient distance beyond its support to form an anchorage for the cantilever. Thus in Fig. 81 we have a beam

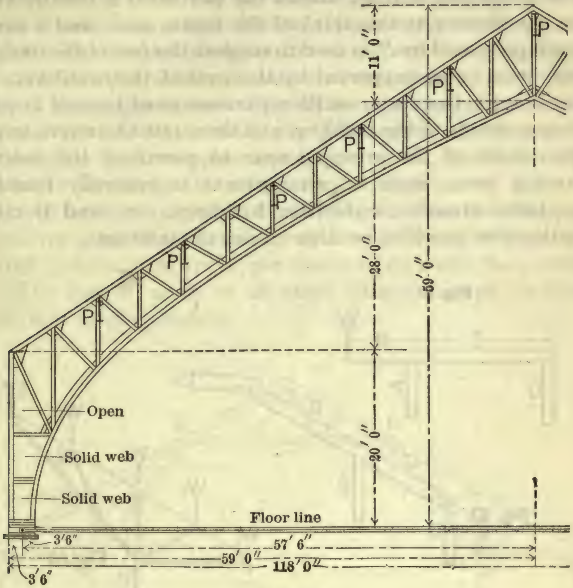


Fig. 80

resting on two supports; the portion B is a cantilever, while the part C forms the anchorage for it.

(In applying the cantilever to trusses it is customary to interpret it as including both the projecting arm and the balancing arm, as both portions form one piece of framework, and the term will be so used in this work.)

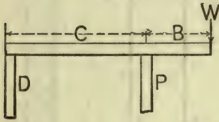
It is obvious that if the entire beam (Fig. 81) were uniformly loaded the post P would carry the greater part of the weight, and also that an additional load at W might produce an upward pull on the post D, in which case the stress on P would exceed the load on the beam.

Both conditions of loading occur in practice, although it probably most often happens that the outer end of the truss requires anchorage rather than a support.

As applied to roof construction some such arrangement as is shown in Fig. 82 is generally required to make this method of support practicable; that is, a wide centre span, with shorter spans or aisles on each side of it.

The projecting or inner arm of the cantilever is usually made from one-quarter to one-third of the centre span, and a simple truss, represented by *S*, is used to support the rest of the roof, the centre truss being supported by the arms of the cantilever. In all such cases, therefore, cantilever trusses must be used in pairs, one on each side of the building, and there must be rooms or passages outside of the principal span to permit of the outer or balancing arm. Such an arrangement is generally found in large halls, armories, exhibition buildings, etc., and it might sometimes be provided in other classes of buildings.

Fig. 81



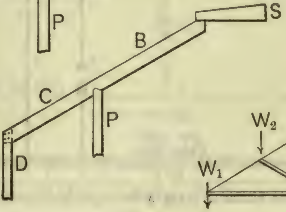


Fig. 82

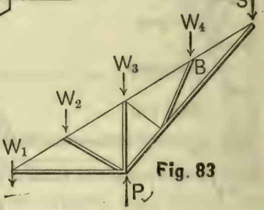
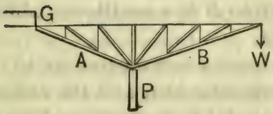


Fig. 83



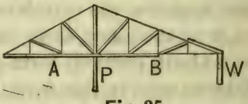


Fig. 84 **Fig. 85**

Of course, in a large building a simple beam such as is shown in Fig. 82 could not be used, but the principle of construction is the same whether the cantilever be a simple beam or a large truss.

Fig. 83 shows the diagram for a truss to take the place of the beam *CB*, Fig. 82, the single lines representing the tension

members and the double lines compression members, and Fig. 86 shows the complete arrangement of the trusses.*

The truss shown in these figures may be extended to almost any extent, and the lower chord may be curved, but the general outline of the truss will be found best adapted for all cases where a wide central roof is to be supported by cantilevers.

For bridge trusses or floors the shape shown in Fig. 84 may be used, and for shed and platform roofs open on one side a truss of the shape shown in Fig. 85 is about the only practicable device. In this latter truss the proportions of the arms are such that a slight support is required at W, thereby bringing the lower portion of the rafter into compression.

It will be seen from Figs. 83, 84, and 85 that the strains in a cantilever truss are directly the reverse of those in trusses supported at both ends, the upper chord or rafter in the cantilever being in tension, while in all other trusses, except the hinged arch, it is in compression.

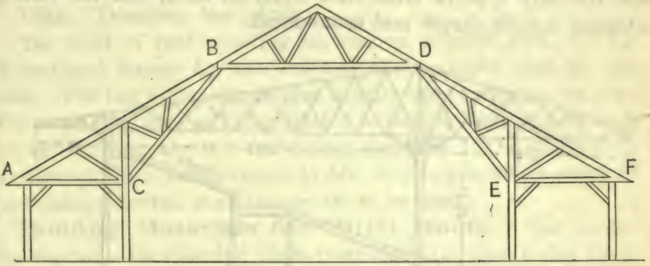


Fig. 86
Suggestion for Wooden Cantilever Truss.

Advantages and Disadvantages of the Cantilever Truss.—The special advantages possessed by the cantilever truss are: A greater clear height in the centre than can be obtained with any other type excepting the three-hinged arch, a light and graceful appearance, no horizontal thrust, and consequently no tie-rods required. The particular advantage of this truss for very great spans is that it can be erected with-

* Suggested by Mr. John Beverly Robinson for a simple trussed cantilever roof.

out scaffolding under the centre, and in bridge work this is considered as its only advantage.

It is claimed by prominent engineers that the cantilever is not an economical type of truss, and not as desirable for spans of 150 feet or more as the three-hinged arch.

It also does not permit of as readily overcoming expansion and contraction as either the three-hinged arch, the bowstring truss, or the quadrangular truss. For certain classes of buildings, however, and especially where the central span does not exceed 150 feet, it can perhaps be used with better architectural effect than is possible with other types, and with about the same economy. For roofing platforms, grand-stands, etc., where an outer support is not desired, it is the only type available.

Example of a Cantilever Truss.—Fig. 87 is a diagram of one of the cantilever trusses supporting the roof of the grand-stand at the Monmouth Park (N. J.) racing-track, the details of which were published in *Architecture and Building* in February, 1890. This is an instance where the cantilever was the only type of truss that could be used, and the form adopted is both simple and economical.

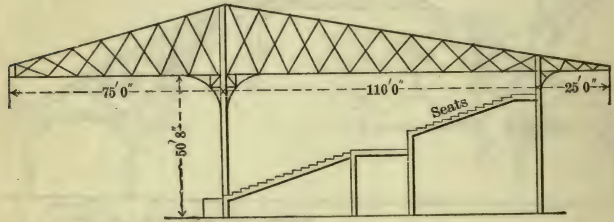


Fig. 87

As will be seen from the drawing, the main supporting post extends to the top of the truss, as is usually the case with cantilever trusses, and the truss is riveted to each side of it. The upper and lower chords were made of two angles and a web-plate, the upper chords or rafters acting as a tie-beam between the bracing. The bracing consists of angle-bars used in pairs and varying from $3 \times 2 \times \frac{1}{4}$ inches to $3 \times 3 \times \frac{5}{16}$ inches, the whole frame being connected by rivets. Other examples of cantilever roofs are given in Part III of "Building Construction."

CHAPTER XXVI.

STRESSES IN ROOF-TRUSSES.

THE various steps to be pursued in designing a trussed roof and proportioning its parts are as follows:

First. Deciding upon the roof covering and how it is to be supported between the trusses.

Second Laying out the roof trusses on the plan and section.

Third. Computing the truss loads and determining the stresses produced thereby.

Fourth. Computing the size of the truss members.

Fifth. Detailing the joints.

The kind of roof covering to use, on a pitch roof, will be determined largely by the external effect sought, and by the cost. For flat roofs, appearance usually cuts no figure, so that the durability, cost, and adaptability to any peculiar requirements of the building are the controlling elements. The matter of incombustibility, or resistance to fire, is also generally a point to be considered when steel trusses are to be used.

Roofing Materials for Pitch Roofs.—The materials suitable for covering pitch roofs are slate, burnt clay tiles, metal tiles or shingles, wood shingles, corrugated iron, tin with standing seam, standing seam steel roofing, and various kinds of ready roofing.

The least slope to which these materials may be laid without danger of leaks, the weight per square foot of roof and the comparative cost is indicated by Table I. The cost, however, can only be considered as approximate as it will vary for different materials, according to the locality, and the scale of wages.

Flat roofs or roofs having a fall of ½ in. to ⅝ in. to the foot are usually covered with tar and gravel, asphalt, ready roofing, or tin with lock-and-solder joint. A good tin roof costs about $8 a square, besides the painting. The other kinds will vary from $3.50 to $4.50 a square.

TABLE I.—COVERING MATERIALS FOR PITCH ROOFS.

Material	Least Rise of Rafter in 12 ins.	Comparative Cost per Square.
Slates, black	8	\$7 to \$12
Slates, green	8	\$7 to \$10
Slates, red	8	\$12 to \$17
Burnt clay tiles, interlocking pattern	7	\$15 to \$25
Tin shingles, painted	6	\$8 to \$10
Galvanized iron tile, painted	6	\$13 to \$15
Cedar shingles, stained or painted	6	\$3.80 to \$7.20
Corrugated iron, painted	3	\$4.00 to \$4.50
Standing seam steel roofing, painted	2	\$4 to \$4.50
Ready roofing	1	\$3.50 to \$4.50

Manner of Supporting the Roof from the Trusses. —Wooden roofs, supported by wooden trusses, require common or jack rafters to support the sheathing or slate, and generally purlins to support the rafters, although in some cases it may be more economical to span the rafters from truss to truss, see p. 891.

When slate or burnt clay tile are used on steel roofs, they are usually secured to steel angles, running parallel with the walls and spaced from 8 to 10½ ins. apart, as may be necessary to accommodate the size of the slate or tile. If the span is not very great, the angles may be fastened to the truss rafters, which will require that the trusses be not more than 6 or 7 ft. apart. As a rule, however, when slate or tile are to be used, it will be cheaper to space the trusses from 16 to 20 ft. apart, and to use purlins and jack rafters for supporting the smaller angles.

Quite often, wooden rafters and sheathing are used with steel trusses; this is more economical, but, of course, increases the fire risk, unless there is a wooden ceiling below, in which case unprotected steel is little if any better than wood.

If corrugated iron is to be used for roofing, the most economical construction for steel roofs will be to space the trusses from 16 to 20 ft. apart, and to use light I-beams for purlins, spaced about 4′ 9″ on centers, as in Fig. 60, Chapter XXV, the corrugated iron being secured to the purlins by straps. If warm air comes in contact with the underside of a corrugated roof

the roofing should be laid on boards, or some kind of anti-condensation lining provided, otherwise the moisture in the air will condense and fall on the floor or objects below.

Flat roofs will always require rafters and sheathing, or fireproof filling between the rafters.

Spacing of Trusses.—From the above it will be seen that the economical spacing of the trusses will depend to a considerable degree upon the kind of roofing that is to be used, and also upon the span. As a general rule, however, the most economical spacing will be about as follows:

For wooden trusses under 80 ft. span, 12 to 16 ft. C. to C.

For wooden trusses over 80 ft. span, 16 to 24 ft. C. to C.

For steel trusses under 80 ft. span, 16 to 20 ft. C. to C.

For steel trusses over 80 ft. span, 20 to 40 ft. C. to C.

The actual spacing of a number of steel trusses of wide span is given in Table VI.

When the distance between the trusses exceeds 16 ft. for wooden roofs or 20 ft. for steel roofs, it will generally be necessary to use trussed purlins.

Having decided upon the kind of truss to be used, the spacing of the trusses and the roof construction, a section drawing of the roof should be made, showing an elevation of the truss, the points at which the purlins are to be supported, and also the manner of supporting the ceiling, if any, and any other loads that are to be supported by the trusses.

The section and truss drawing with a knowledge of the weight of materials, will afford the necessary data for computing the loads at each joint of the truss.

Until the stresses have been determined, the size of the members computed, and the joints detailed, an exact drawing of the truss cannot, of course, be made, but to compute the loads and stresses, it is necessary to know the position of the joints, and these can be indicated with sufficient accuracy without knowing the exact size of the members. Chapter XXV gives sufficient information regarding the various types of trusses, to enable one to decide on the height, and the number and position of the struts and ties, and one can guess at the size of the members for the preliminary drawings.

Roof and Ceiling Area Supported at any Joint.

Calculations for the stresses in a truss are always based on the assumption that the loads are transferred to the joints, and that the members are free to move at the joints as if hinged, even although the actual joint may be made with riveted connections. The loads at the joints are, of course, equal to the reaction of the purlins, or of the tie-beams or principals, if these receive the ceiling joists or rafters. When the load on the roof or ceiling is uniformly distributed, as is usually the case, the simplest method of computing the joint loads, is to find the roof or ceiling area contributory to the joint, and multiply this area by the weight or load per square foot.

As a rule, the area contributory to any joint is equal to the distance half way to the next joint on each side, multiplied by the distance half way to the next truss or wall, on each side. Thus if Fig. 1 represents truss 1, of Fig. 2, the roof area contributory to joint 2, is $\frac{8+14}{2}\times a$. For truss 2, the area supported by the same joint is $\frac{14+12}{2}\times a$, or if we let D represent the

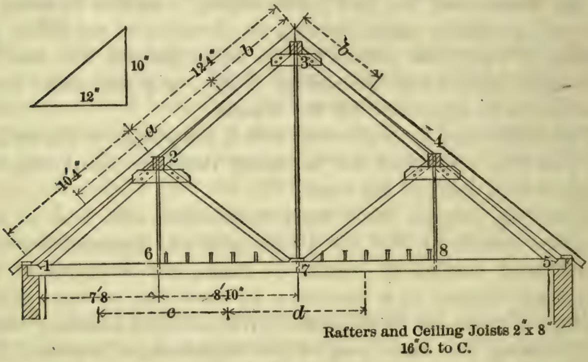


Fig. 1

length of roof or ceiling supported at each joint, then the area supported by joint $2 = a \times D$ and the area supported by joint $3 = 2b \times D$. In the same way, the ceiling area supported at joint $6 = c \times D$, the arrow-heads being half way between the joints.

It makes no difference in the joint loads whether the common rafters are supported on purlins or whether they rest on the

rafter provided the purlins come at or close to the joints and the load is uniformly distributed.

Thus the width of the ceilling contributory to joint 7, Fig. 3, will be equal to c, just the same as in Fig. 1, but it makes a considerable difference in the strain in the tie-beam. When the trusses are spaced a uniform distance apart, D, Fig. 2, will, of course, be equal to the distance between centres of trusses. When the trusses are not spaced uniformly, D equals one half the distance from the centre of the truss on the left to the centre of the truss on the right.

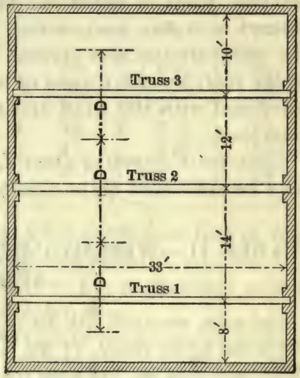


Fig. 2

When the purlin comes more than 12 ins. from a joint, or the roof area is not symmetrical, as is often the case at hips and valleys, then the joint load must be determined by the principle of the reaction of beams, as explained on pp. 274–277. Examples showing the computation of joint loads are given a little further on.

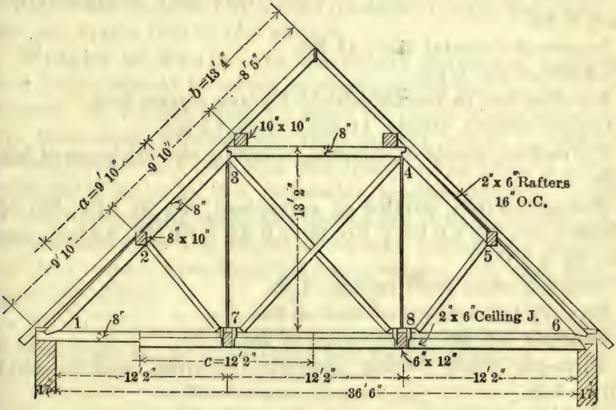


Fig. 3

Roof Load per Square Foot.—By the term "roof load" is meant the weight of the materials composing the roof, trusses, and purlins, an ample allowance for snow and also an allowance for wind pressure. The weight of the materials compose what is called *the dead load.* Snow is generally considered as a live load, acting vertically. The pressure due to the wind always acts normal, or at right angles to the surface of the roof, but for trusses of less than 100 ft. span it is usually combined with the wind and snow loads and treated as a vertical load.

Data for Computing Dead Loads.—The dead load of any roof may be estimated quite closely from the following data:

TABLE II.—WEIGHTS PER SQUARE FOOT OF ROOF SURFACE.

Shingles, common, $2\frac{1}{2}$ lbs.; 18 ins., 3 lbs.

Slates, $\frac{3}{16}$ in. thick, $7\frac{1}{4}$ lbs.; $\frac{1}{4}$ in. thick, 9.6 lbs. (the common thickness is $\frac{3}{16}$ in. for sizes up to $10'' \times 20''$).

Plain tiles or clay shingles, 11 to 14 lbs.

Roman tiles, old style, two parts, 12 lbs.; new style, one part, 8 lbs.

Spanish tiles, old style, two parts, 19 lbs.; new style, one part, 8 lbs.

Improved Oriental tiles, 11 lbs.

Ludowici tile, 8 lbs.

For tiles laid in mortar add 10 lbs. per square foot.

Copper roofing, sheets, $1\frac{1}{2}$ lbs.; tiles, $1\frac{3}{4}$ lbs.

Tin roofing, sheets or shingles, including one thickness of felt, 1 lb.

Corrugated iron, painted or galvanized, No. 26, 1 lb.; No. 24, 1.3 lbs.; No. 22, 1.6 lbs.; No. 20, 1.9 lbs.; No. 18, 2.6 lbs.; and No. 16, 3.3 lbs.

Standing seam steel roofing, 1 lb.

Five-ply felt and gravel roof, 6 lbs.

Four-ply felt and gravel roof, $5\frac{1}{2}$ lbs.

Three-ply ready roofing (elaterite, ruberoid, asphalt, etc.), 0.6 to 1 lb.

Skylights with galvanized iron frame, $\frac{1}{4}$-inch glass, $4\frac{1}{2}$ lbs.; $\frac{5}{16}$-in., 5 lbs.; $\frac{3}{8}$-in., 6 lbs.

Sheathing, 1 in. thick, 3 lbs. per square foot for white pine, spruce, or hemlock; 4 lbs. for yellow or pitch pine.

TABLE III.—WEIGHT OF RAFTERS PER SQUARE FOOT.

Size of Rafters, Inches.	Spruce, Hemlock, White Pine, Spacing in Inches, Centre to Centre.			Hard Pine, Spacing in Inches. Centre to Centre.		
	16	20	24	16	20	24
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
2×4	$1\frac{1}{2}$	1.2	1	2	1.6	$1\frac{1}{3}$
2×6	$2\frac{1}{4}$	1.8	$1\frac{1}{2}$	3	2.4	2
2×7	$2\frac{5}{8}$	2.1	$1\frac{3}{4}$	$3\frac{1}{2}$	2.8	$2\frac{1}{3}$
2×8	3	2.4	2	4	3.2	$2\frac{2}{3}$
2×10	$3\frac{3}{4}$	3	$2\frac{1}{2}$	5	4	$3\frac{1}{3}$

Wooden purlins will weigh about 2 lbs. per square foot of roof surface when the span is between 12 and 16 ft.

For steel roofs the size and weight of the purlins and rafters should be computed for each particular case.

For a rough approximation the weight of steel trusses, purlins, and bracing in a roof covered with corrugated iron with no ceiling will run from 4 to 6 lbs. per square foot of horizontal surface covered. The steel work for slate roofs with suspended ceilings below will run about $7\frac{1}{2}$ lbs. per square foot when the span does not exceed 50 ft.

Steel roofs supported by arched trusses will weigh from 8 to 12 lbs. per square foot of roof surface.

Weight of Truss.—To the weight of the roof construction proper should be added an allowance for the weight of the trusses. If trusses could be built in exact accordance with the theoretical requirements their weight would be directly proportional to the roof load and span, but as there is always some extra material, it is impossible to determine the weight of the truss exactly until the trusses are completely designed. Several tables for the weight of wooden trusses and formulas for steel trusses have been published, but hardly any two of them are alike.*

* The following are some of the formulas given for weight of steel trusses, W being weight per horizontal square foot, S = span in feet.

Charles Evan Fowler, C.E., for Fink trusses:

$W = .06S + .6$ for heavy loads;

$W = .04S + .4$ for light loads.

H. G. Tyrrell, C.E.:

$$W = .05S + \frac{12}{\text{dist. centre to centre}}$$

C. W. Bryan, C.E.:

$$W = .04S + 4.$$

Tables IV and V compiled by the author, from a comparison of other tables and formulas, and from the weight of actual trusses, are sufficiently accurate for the purpose of determining stresses. The weights given are probably slightly in excess of the actual weights of average trusses, as the author prefers to have the error, if any, on the safe side. It should be noted that the weights are for each square foot of roof surface, and not for the horizontal area. Table VI gives the actual weights of a number of large steel roofs.

TABLE IV.—WEIGHT PER SQUARE FOOT OF ROOF SURFACE FOR WOODEN TRUSSES.*

Span.	½ Pitch.	⅓ Pitch.	¼ Pitch.	Flat.
	lbs.	lbs.	lbs.	lbs.
Up to 36 ft.	3	3½	3¾	4
36 to 50 ft.	3¼	3¾	4	4½
50 to 60 ft.	3½	4	4½	4¾
60 to 70 ft.	3¾	4½	4¾	5¼
70 to 80 ft.	4¼	5	5½	6
80 to 90 ft.	5	6	6½	7
90 to 100 ft.	5¾	6¾	7	8
100 to 110 ft.	6½	7½	8	9
110 to 120 ft.	7	8½	9	10

* For scissors trusses increase one-third.

TABLE V.—WEIGHT PER SQUARE FOOT OF ROOF SURFACE FOR STEEL TRUSSES.

Span.	½ Pitch.	⅓ Pitch.	¼ Pitch.	Flat.
Up to 40 ft.	5.25	6.3	6.8	7.6
" 50 ft.	5.75	6.6	7.2	8.0
" 60 ft.	6.75	8.0	8.6	9.6
" 70 ft.	7.25	8.5	9.2	10.2
" 80 ft.	7.75	9.0	9.7	10.8
" 100 ft.	8.5	10.0	10.8	12.0
" 120 ft.	9.5	11.0	12.0	13.20
" 140 ft.	10.0	11.6	12.6	14.0

The data for the first seven buildings in Table VI were compiled by Mr. H. G. Tyrrell, C.E., who states that all of the seven roofs were proportioned for slate and plank roofing resting on wide rafters 2 ft. apart, supported by steel purlins about 10 ft. apart. Spans given are centre to centre of side bearings.

Stresses computed for dead load of 25 lbs., snow load of 10 lbs. per square foot of sloping surface, horizontal wind 40 lbs. per square foot or 28 lbs. normal pressure.

Data for computing the weight of floors and floor loads supported by trusses, or for fireproof construction, may be found in Chapter XXI and XXIII.

TABLE VI.—WEIGHT AND SPACING OF SOME STEEL ROOFS OF WIDE SPAN, INCLUDING TRUSSES, PURLINS, AND BRACES, BUT NOT ROOF COVERING OR RAFTERS.

Name of Building.	Type of Truss.	Span, Feet.	Spacing C. to C. of Trusses.	Wt. per Sq. Ft. Sloping Surface.	Wt. of of One Truss.
			Feet	Lbs.	Tons
Pawtucket Armory	Fig. 80*	82	24	8.7	6.7
Portland, Me., Armory	"	92	25	9.7	9
Phœnix Hall, Brockton	"	96	24	8.6	10
Northampton Armory	"	100	24	8.0	8.5
Palace Rink, Hartford	"	104	25	11.8	11.5
Providence Ex. Hall	"	118	24½	9.5	12.5
Cleveland Armory	"	120	23–25		
Boston Armory	"	122	30	12.4	21
22d Regt. (N. Y.) Armory	"	176	24½		
Brooklyn Armory	"	196	35		
Kansas City Convention Hall	Fig. 73*	187⅓†	23½		
74th Reg't Armory, Buffalo	3-hinge arch	227†	28		
Chicago Coliseum	3-hinge arch	149¾†	23–25		

* Chapter XXV. † Centre to centre of end pins.

Snow.—As a basis for making an allowance for snow, Table VII is perhaps as good a guide as any that can be given. When *snow guards* are to be placed on a roof, the same allowance should be made for a half pitch as for one-third pitch.

TABLE VII.—ALLOWANCE FOR SNOW IN POUNDS PER SQUARE FOOT OF ROOF SURFACE.

Location.	Pitch of Roof.				
	½	⅓	¼	⅕	⅙ or less.
	* †	* †	* †		
Southern States and Pacific Slope	0–0	0–5	0–5	5	5
Central States	0–5	7–10	15–20	22	30
Rocky Mountain States	0–10	10–15	20–25	27	35
New England States	0–10	10–15	20–25	35	40
Northwest States	0–12	12–18	25–30	37	45

Columns headed by an asterisk (*) are for slate, tile, or metal; those headed by a dagger (†) are for shingle roof.

Wind Pressure.—For roofs having a pitch of 5 ins. or more to the foot, an allowance must be made for wind pressure. For trusses of the Fink, Fan, King, or Queen types the usual practice is to include the wind pressure with the vertical loads, and to make a single allowance for both wind and snow, as during a gale snow is not likely to stay on a steep roof.* When the wind pressure is added to the vertical loads, the author recommends that the allowance for wind and snow combined be not less than indicated in Table VIII.

TABLE VIII.—ALLOWANCE FOR WIND AND SNOW COMBINED IN POUNDS PER SQUARE FOOT OF ROOF SURFACE.

Location.	Pitch of Roof.					
	60°	45°	1/3	1/4	1/5	1/6
Northwest States	30	30	25	30	37	45
New England States	30	30	25	25	35	40
Rocky Mountain States	30	30	25	25	27	35
Central States	30	30	25	25	22	30
Southern and Pacific States	30	30	25	25	22	20

No roof truss should be proportioned for a total load of less than 40 lbs. per square foot, except flat roofs in warm climates.

For trusses having a span exceeding 100 ft. (except horizontal trusses) and for trusses in which a partial load may produce maximum stresses, or call for counter bracing, as is the case in quadrilateral trusses, and trusses with curved chords the stresses for all the different loadings should be found separately and each member of the truss proportioned to the maximum stress to which it may be subject under any possible combination of the load.

For determining the stresses due to wind pressure alone the force of the wind is usually assumed to act in a direction normal, i.e., at right angles to the slope of the roof. This force is commonly based on a horizontal wind pressure of 30 lbs.

* Mr. Bryan, the designing engineer of the Edgemoor Bridge Works, states that "in Fink trusses a partial load due to wind or snow never causes any maximum stresses, so that it is customary to calculate these trusses for a uniform load over the entire truss, the wind and snow loads combined being usually assumed at 30 lbs. per sq. ft. of area covered; i.e., horizontal surface."

per square foot, although quite often it is taken at 40 lbs. per square foot, depending somewhat upon the exposure and the shape of construction of the roof and truss.

The normal and horizontal pressure per square foot of roof surface corresponding to a horizontal pressure of 30 lbs. against a vertical surface is given in Table IX.

TABLE IX.—NORMAL AND HORIZONTAL WIND PRESSURE ON ROOFS FOR 30 POUNDS HORIZONTAL PRESSURE AGAINST A VERTICAL SURFACE.

Inclination.	Norm.	Hor.	Inclination.	Norm.	Hor.
	lbs.	lbs.		lbs.	lbs.
5°	3.9	0.3	30°	19.9	10.0
10°	7.2	1.2	33°–41′ (1/3 pitch)	22.0	12.0
15°	10.5		35°	22.6	
18°–26′ (1/6 pitch)	13.0	4.0	40°	25.1	15.9
20°	13.7	4.5	45° (1/2 pitch)	27.1	19.0
21°–48′ (1/5 pitch)	15.0	6.0	50°	28.6	21.9
25°	16.9		55°	29.7	
26°–34′ (1/4 pitch)	18.0	8.0	60°	30.0	25.5

For horizontal wind pressure of 40 lbs. per square foot the pressure given above should be increased one-third.

Variations in Loading for which Stresses should be Found.—To determine the maximum stresses under any possible condition of loading, stresses should be found for the following cases:

(1) Stresses due to permanent dead loads.

(2) Snow covering only one side of roof.

(3) Snow covering entire roof.

(4) Wind on side of truss nearer the expansion end.

(5) Wind on side of truss nearer the fixed end.

It is generally assumed that the maximum wind pressure and the snow load cannot act on the same half of the truss at the same time, hence the combinations for maximum stress will be either cases 1 and 3 or cases 1, 2, and 4 or 5.

If the trusses are supported on iron columns instead of walls the wind force is transferred to the foundations through the columns, producing a bending moment in the columns. The strains in the columns, trusses, and knee-braces should therefore be determined for the horizontal wind pressure against the side of the building and roof. This pressure is obtained by multiplying the area of the vertical surfaces by the full

pressure per square foot and the elevation of the roof by the horizontal component, given in Table IX.

For the trusses supporting the roof of the Kansas City Auditorium (see Fig. 73, Chapter XXV) stresses were computed for the following conditions: First, full dead and live load on both galleries and the roof-garden and wind pressure due to a velocity of 45 miles an hour; second, full dead load, snow load, and gallery live load, wind pressure 10 lbs. and no load on roof-garden floor; third, full dead load and 50 lbs. wind pressure; fourth, full dead load and wind pressure at 45 miles an hour, and full live loads on gallery and roof-garden on one side only.

Snow loads throughout were taken at one-third of the dead load. Examples showing manner of combining the stresses due to different conditions of loading are given on pp. 1023 and 1034.

Examples of the Computation of Roof Loads.

(All loads considered as acting vertically.)

1. For the first example we will take the roof and truss shown by Fig. 1 (p. 944), which we will assume represents truss 2 of Fig. 2. We will assume that the timber is to be common white pine and that the roof is to be covered with $\frac{3}{16}$-inch slate of medium size on $\frac{7}{8}$-inch sheathing.

The ceiling to consist of lath and plaster.

The dead load of roof and truss per square foot of roof surface will be made up as follows:

For slate	$7\frac{1}{4}$	lbs.
For sheathing	3	"
For rafters	3	"
For purlins	2	"
For truss	3	"
Total	$18\frac{1}{4}$	lbs.

For wind and snow load combined we should allow about 28 lbs. (the pitch being about 40 degrees), which would make a total roof load of $46\frac{1}{4}$ lbs. To avoid fractions, however, we will take 48 lbs. per square foot.

As the distance to truss 1, Fig. 2, is 14 ft., and to truss 3 12 ft., the length of roof supported by the truss will be 13 ft. The roof area supported by the purlins at joint 2 will be equal to the distance *a* multiplied by 13 ft., and *a* will be one-half of the distance from the wall plate to the next purlin, or 22′ 8″ ÷ 2

$=11' \ 4''$, or $11\frac{1}{3}$ ft. Hence the roof area supported at joint 2 will be $11\frac{1}{3} \times 13$, or $147\frac{1}{3}$ square feet.

The roof area supported by the purlins at joint 3 will be $2b \times 13$ ft., or $12' \ 4'' \times 13' = 160\frac{1}{3}$ square ft.

Multiplying the roof areas by the load per square foot (48), we have 7,072 lbs. for the load at joint 2 and 7,696 lbs. for the load at joint 3. The load at joint 4 will be equal to that at 2, as the truss is symmetrical.

We must now compute the ceiling loads at joints 6 and 7. The ceiling area supported at joint $6 = c \times 13$ ft., or $8\frac{1}{4} \times 13 = 107\frac{1}{4}$ sq. ft. The area supported at joint $7 = 8\frac{5}{8} \times 13 = 114\frac{5}{8}$ sq. ft.

The actual weight of the ceiling per square foot will be 3 lbs. for the joists and 10 lbs. for lath and plaster; but where there is a large attic space it is liable to be used for storing odd articles, so that it is always well to make a small allowance, say 5 lbs. per square foot, for any extra weight that might be placed in the attic. We will, therefore, allow 18 lbs. per square foot for the weight of the ceiling, which would make the weight at joints 6 and 8 $107\frac{1}{4} \times 18$, or 1,930 lbs., and the weight at joint 7 $114\frac{5}{8} \times 18 = 2{,}067$ lbs.

As soon as computed, the roof and ceiling loads should be marked on a truss diagram, as in Fig. 10. The roof and ceiling loads at joint 1 are transmitted directly to the wall and need not be taken into account in determining the stresses.

Example 2.—To compute the joint loads for the truss shown by Fig. 3, p. 945, all timber to be of spruce and the roof to be covered with shingles on 1-inch sheathing; the ceiling to be of lath and plaster.

For the dead load per square foot we have

Weight of shingles	$2\frac{1}{2}$	lbs.
Weight of sheathing	3	"
Weight of rafters	$2\frac{1}{4}$	"
Weight of purlins	2	"
Weight of truss	3	"
Total dead load per square foot	$12\frac{3}{4}$	lbs.
Allowance for wind and snow	30	lbs.
Total roof load per square foot	$42\frac{3}{4}$	"

For the weight of the ceiling it will be well for a truss of this kind to allow at least 20 lbs. per square foot.

We will assume that the trusses are to be spaced uniformly 15 ft. centre to centre.

Then the roof area supported at joint 2 will be 9′ 10″×15′, or 147½ square feet, and the load at this joint 6,306 lbs. The purlin at joint 3 supports the roof, from a point midway to joint 2, to the ridge, or $b = 4' \ 11'' + 8' \ 5''$, or 13′ 4″. The roof area supported at this point is 13′ 4″×15′, or 200 sq. ft., and the load 8,550 lbs.

The loads at joints 4 and 5 will be equal respectively to those at 3 and 2.

For the ceiling loads at joints 7 and 8 we have an area to be supported = 12′ 2″×15′, or 182½ sq. ft., which multiplied by 20 gives 3,650 lbs.

EXAMPLE 3.—For this example we will take the church roof shown in section by Fig. 4. In this roof the trusses take the

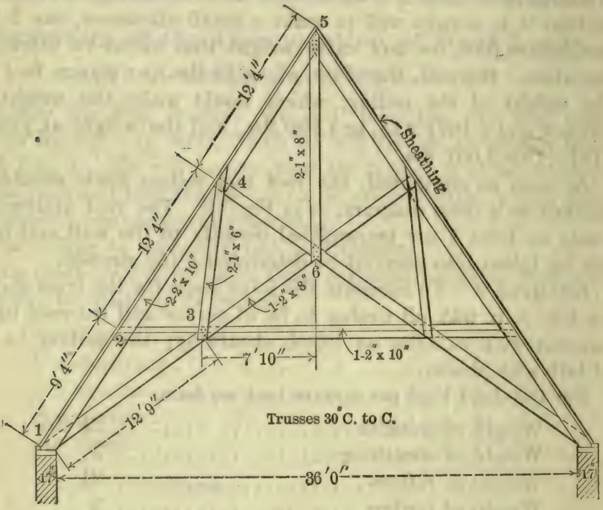


Fig. 4

place of the rafters and ceiling beams, the sheathing spanning from truss to truss and the laths for the ceiling being nailed to 1¼″×2½″ furring strips spaced 12 or 16 ins. on centres. Assuming that the parts of the trusses will have the dimensions indicated in the figure, and that the wood is to be white pine, the actual weight of one truss will be about 1,200 lbs. The roof area supported by one truss is 170 sq. ft., hence the weight of the trusses will be about 7 lbs. per square foot of

roof surface. (Note. It will be seen that this weight is more than twice that given in Table IV, owing principally to the trusses being so close together and the members of small dimensions.) The weight of the sheathing and shingles will be about $5\frac{1}{2}$ lbs., and we will allow 30 lbs. for wind pressure. (The roof is too steep for snow to lodge on it.)

This gives us a total roof load of $42\frac{1}{2}$ lbs. per square foot of sloping surface. For the weight of the ceiling 12 lbs. per square foot will be ample, as no load other than its own weight is likely to come upon it.

The roof area supported at joint $2 = 10\frac{5}{8}' \times 2\frac{1}{2}'$, or 27 sq. ft. The area supported at joints 4 and 5 is equal to $12\frac{1}{3}' \times 2\frac{1}{2}' = 31$ sq. ft. for each. Ceiling area supported at joint $3 = 14\frac{1}{8}' \times 2\frac{1}{2}'$, or $35\frac{1}{2}$ sq. ft. Multiplying the joint areas by the corresponding loads per square foot we have 1,148 lbs. for the load at joint 2, 1,318 lbs. for the load at joints 4 and 5, and 426 lbs. for the load at joint 3.

EXAMPLE 4.—Roof corrugated iron, supported by a steel truss of the shape shown by Fig. 60 of Chapter XXV. This truss supports nothing but the corrugated iron and the purlins and the pressure due to wind and snow, the purpose for which the building is used being such that there will be no occasion for suspending any load from the trusses.

In figuring the dead loads for such a roof, the size of the purlins and the gauge of the iron should first be definitely fixed, so that the weight per square foot of roof may be accurately determined. In this instance the purlins are 5-inch I beams spaced 4′ 9″ centre to centre and weighing 10 lbs. per lineal foot.

The weight of the purlins per square foot of roof is therefore equal to 10 lbs. divided by $4\frac{3}{4}$, or 2.1 lbs.

For a span of 4′ 9″ the corrugated iron should be No. 20 gauge (see Corrugated Iron, Part III), weighing 1.9 lbs. per square foot.

For the weight of the truss and bracing we will take the weight given in Table V for a span of 100 ft. and $\frac{1}{4}$ pitch, 10.8 lbs.* This will give us a total dead load of 14.8 lbs. per square foot of sloping surface.

For wind and snow we should allow 22 lbs. per square foot

* The actual weight of this truss and bracing was 4 lbs. per square foot of sloping surface, which is remarkably small.

if the building is situated in the Central States, making the total roof load 36.8 lbs. per square foot. It is quite generally recommended, however, that no roof should be designed for a less load, all told, than 40 lbs. per square foot; therefore the joint loads should be computed on that basis.

The only loaded joints in this truss are under the purlins, and as the trusses are spaced 19′ 2¼″ centre to centre, and the purlins 4′ 9″ centre to centre, the roof area supported at each upper joint is 91 sq. ft. Hence the joint loads should be figured at 3,640 lbs. (Note. Even for the locality in which it was built, this is a very light roof and would hardly be safe in the more Northern or Western States.)

EXAMPLE 5.—Flat roof (Fig. 5). Timber to be of spruce; five-ply gravel roof and plastered ceiling.

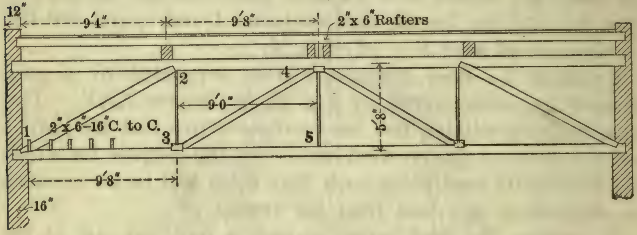


Fig. 5

For the dead load we have

Weight of roofing	6	lbs.
" " sheathing	3	"
" " rafters	2¼	"
" " purlins	2	"
" " truss, say	4¼	"
Total dead load	17½	lbs. per sq. ft.

No allowance will be required for wind pressure, but the snow load will be a considerable item in any of the Northern States, as indicated in Table VII. Assuming that the building is located in one of the Central States, we should allow 30 lbs. per square foot for snow, making the total roof load 47½ lbs. The plaster ceiling and the ceiling joists will weigh about 12¼ lbs., and as the roof space is not likely to be used for storage, 13 lbs. per square foot will be a sufficient allowance for the ceiling.

Assuming that the trusses are to be uniformly spaced, 14 ft. centre to centre, the roof area supported at joint 2 will be $9\frac{1}{2}' \times 14'$, or 133 sq. ft., and the area supported at joint 4, $9\frac{2}{3}' \times 14'$, or $135\frac{1}{3}$ square feet.

The ceiling area supported at joint 3 will be $9\frac{1}{3}' \times 14'$, or $130\frac{2}{3}$ sq. ft., and at joint 5, $9' \times 14'$, or 126 sq. ft.

Multiplying these areas by the corresponding loads per square foot, we have 6,317 lbs. for the load at joint 2, 6,428 lbs. at joint 4, 1,699 lbs. at joint 3, and 1,638 lbs. at joint 5.

In practice it is hardly worth while to try to compute the stresses closer than 100 lbs., so that the loads may as well be put down at an even 50 or 100 lbs. above the load obtained by computation. When the roof is supported by purlins, there are often some joints of the truss which have no load. Thus for the truss shown by Fig. 19, Chapter XXV, there would be no loads on joints 2, 6, and 10.

The roof area supported at joint 4 (Fig. 19) is equal to one-half the distance *OB* multiplied by the distance halfway to the truss on each side. If the lower chord supports ceiling joists, then there will be a load at each of the joints 3, 5, 7, 9, etc. Stress diagrams can be drawn for any arrangement of loads, the important point being to compute the loads exactly as they will be imposed on the truss. These five examples illustrate fairly well the method of computing the loads on a truss. Special cases of loading should be computed on the same principle.

Determining the Stresses.

To determine the stresses, a diagram of the truss, composed of single lines representing the centre line of the truss members, should first be carefully drawn to a scale and the loads at the different joints indicated by arrows and numbers as in Figs. 10 and 12. If the centre lines of the members as they are actually placed do not intersect at common points, they must be made to do so in the diagram, as the stresses can be computed only on the assumption that the centre lines of all members meeting at any point intersect at a common point.

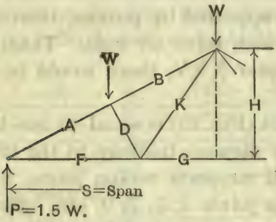
In wooden trusses it is not always practicable to place the braces so that their centre line will pass through the centre of the joints, but they should come as near to it as practicable, and in steel trusses the joint connections should be made so that the centre lines of all members meeting at a joint will intersect at the same point.

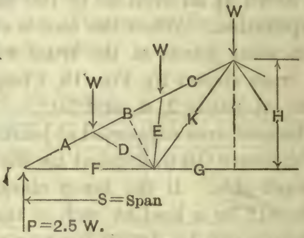
Stresses Obtained by Direct Computations.

As a general rule, the stresses in a roof truss can be determined much more readily by the graphic method than by mathematical computations and with as close a degree of accuracy as is necessary. There are a few forms of trusses, however, for which the stresses can be quite easily determined by computation *provided the truss is perfectly symmetrical*

TABLE X.—COEFFICIENTS FOR DETERMINING THE STRESSES IN SIMPLE FINK AND FAN TRUSSES

When panel loads are all equal.





To find the stress in any member multiply its factor by panel load W.

SIMPLE FINK TRUSS.

Member.	Kind of Stress.	$\frac{S}{H}=3.$	$\frac{S}{H}=3.464 = 30°.$	$\frac{S}{H}=4.$	$\frac{S}{H}=5.$
A	Comp.	2.71	3.00	3.35	4.04
B	"	2.15	2.50	2.91	3.66
D	"	0.83	0.87	0.89	0.93
F	Tension	2.25	2.60	3.00	3.75
G	"	1.50	1.73	2.00	2.50
K	"	0.75	0.87	1.00	1.25

SIMPLE FAN TRUSS.

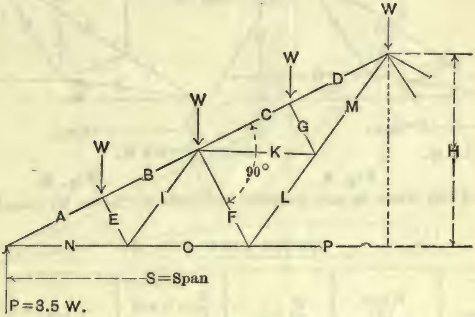
A	Comp.	4.50	5.00	5.59	6.73
B	"	3.53	4.00	4.55	5.58
C	"	3.39	4.00	4.70	5.98
D	"	0.93	1.00	1.08	1.21
E	"	0.93	1.00	1.08	1.21
F	Tension	3.75	4.33	5.00	6.25
G	"	2.25	2.60	3.00	3.75
K	"	1.50	1.73	2.00	2.50

and the joint loads *all alike*, as is quite frequently the case with simple steel roofs having no ceiling load.

Tables X to XIII give constants by which the stresses in Fink and fan trusses may be readily computed simply by multiplying the constant by the panel or joint load. These tables only apply, however, when the rafter is divided by the struts into uniform spaces, giving uniform panel loads. For any

TABLE XI.—COEFFICIENTS FOR DETERMINING THE STRESSES IN EIGHT-PANEL FINK TRUSS

When panel loads are all equal.



To find the stress in any member multiply its factor by panel load W.

Member.	Kind of Stress.	$\frac{S}{H}=3.$	$\frac{S}{H}=3.464$ $=30°.$	$\frac{S}{H}=4.$	$\frac{S}{H}=5.$
A	Comp.	6.31	7.00	7.83	9.42
B	"	5.75	6.50	7.38	9.05
C	"	5.20	6.00	6.93	8.68
D	"	4.65	5.50	6.48	8.31
E	"	0.83	0.87	0.89	0.93
F	"	1.66	1.73	1.79	1.86
G	"	0.83	0.87	0.89	0.93
I	Tension	0.75	0.87	1.00	1.25
K	"	0.75	0.87	1.00	1.25
L	"	1.50	1.73	2.00	2.50
M	"	2.25	2.60	3.00	3.75
N	"	5.25	6.06	7.00	8.75
O	"	4.50	5.19	6.00	7.50
P	"	3.00	3.46	4.00	5.00

other conditions the stresses should be computed by the graphic method. Tables XIV and XV give formulas for computing the stresses in Howe trusses. These formulas, unlike the

TABLE XII.—COEFFICIENTS FOR DETERMINING THE STRESSES IN CAMBERED FINK AND FAN TRUSSES

When panel loads are all equal and the camber equals one-sixth the rise.

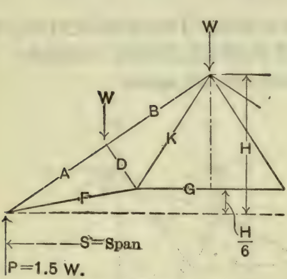


Fig. A

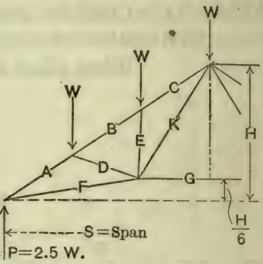


Fig. B

To find the stress in any member multiply its factor by panel load *W*.

TRUSS LIKE FIG. *A*.

Member.	Kind of Stress.	$\frac{S}{H}=3.$	$\frac{S}{H}=3.464$ or 30°.	$\frac{S}{H}=4.$	$\frac{S}{H}=5.$
A	Comp.	3.64	4.13	4.70	5.78
B	"	3.09	3.63	4.25	5.41
D	"	0.83	0.87	0.89	0.93
F	Tension	3.07	3.62	4.24	5.40
G	"	1.80	2.08	2.40	3.00
K	"	1.43	1.69	1.98	2.52

TRUSS LIKE FIG. *B*.

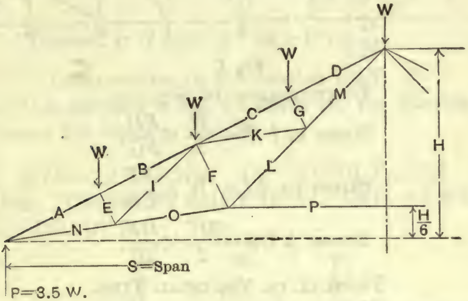
A	Comp.	6.09	6.88	7.83	9.64
B	"	4.89	5.63	6.48	8.10
C	"	4.96	5.88	6.93	8.89
D	"	1.04	1.15	1.26	1.49
E	"	1.04	1.15	1.26	1.49
F	Tension	5.12	6.03	7.07	9.01
G	"	2.70	3.12	3.60	4.50
K	"	2.66	3.13	3.67	4.69

constant in Tables X to XIII, may be used for unequal panel loads provided that the truss is symmetrical about a vertical line drawn half way between the supports.

For the young architect or engineer these tables will be found useful in affording a check upon stresses determined by the graphic method.

TABLE XIII.—COEFFICIENTS FOR DETERMINING THE STRESSES IN EIGHT-PANEL CAMBERED FINK TRUSS.

When panel loads are all equal and camber equals one-sixth the total rise.



To find the stress in any member multiply its factor by panel load W.

Member.	Kind of Stress.	$\frac{S}{H}=3.$	$\frac{S}{H}=3.464$ or 30°.	$\frac{S}{H}=4.$	$\frac{S}{H}=5.$
A	Comp.	8.49	9.63	10.96	13.49
B	"	7.94	9.13	10.51	13.11
C	"	7.39	8.63	10.06	12.74
D	"	6.83	8.13	9.61	12.37
E	"	0.83	0.87	0.89	0.93
F	"	1.66	1.73	1.79	1.86
G	"	0.83	0.87	0.89	0.93
I	Tension	1.02	1.21	1.41	1.80
K	"	1.02	1.21	1.41	1.80
L	"	2.87	3.37	3.96	5.04
M	"	3.89	4.58	5.37	6.85
N	"	7.17	8.44	9.90	12.61
O	"	6.15	7.23	8.48	10.81
P	"	3.60	4.16	4.80	6.00

TABLE XIV.—STRESSES IN TRUSS, FIG. *C*, DUE TO ROOF LOADS ONLY. RAFTERS AND TIE-BEAMS DIVIDED INTO FOUR *EQUAL* SPACES.

W = load at each upper joint.

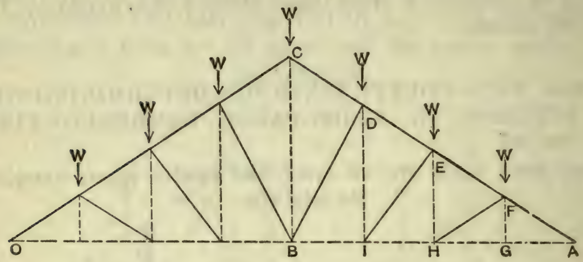


Fig. C

COMPRESSION IN STRUTS.

$$\text{Stress in } FH = \frac{W}{2} \times \frac{FH}{FG}.$$

$$\text{Stress in } EI = W \times \frac{EI}{EH}.$$

$$\text{Stress in } DB = \frac{3W}{2} \times \frac{DB}{DI}.$$

TENSION IN VERTICAL TIES.

Stress in $EH = \frac{W}{2}$; Stress in $DI = W$; Stress in $CB = 3W$.

COMPRESSION IN RAFTER.

$$\text{Stress from } C \text{ to } D = 2W \times \frac{CA}{CB}.$$

$$\text{Stress from } D \text{ to } E = 2\tfrac{1}{2}W \times \frac{CA}{CB}.$$

$$\text{Stress from } E \text{ to } F = 3W \times \frac{CA}{CB}.$$

$$\text{Stress from } F \text{ to } A = 3\tfrac{1}{2}W \times \frac{CA}{CB}.$$

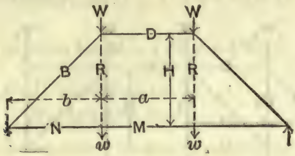
TENSION IN HORIZONTAL TIE.

$$\text{Stress from } B \text{ to } I = 2\tfrac{1}{2}W \times \frac{AB}{CB}.$$

$$\text{Stress from } I \text{ to } H = 3W \times \frac{AB}{CB}.$$

$$\text{Stress from } H \text{ to } A = 3\tfrac{1}{2}W \times \frac{AB}{CB}.$$

TABLE XV.—STRESSES IN SIMPLE QUEEN ROD TRUSS. TRUSS SYMMETRICAL AND SYMMETRICALLY LOADED.



Tension in $R = w$.

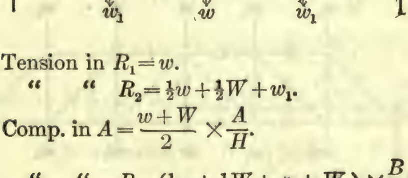
Compression in $B = (w + W) \times \frac{B}{H}$.*

Tension in N and $M = (w + W) \times \frac{b}{H}$.

Compression in D = tension in N.

NOTE.—The distance a has no effect on the stresses, except as it increases the loads w and W.

TABLE XVI.—STRESSES IN FOUR-PANEL HOWE TRUSS. TRUSS SYMMETRICAL AND SYMMETRICALLY LOADED.



Tension in $R_1 = w$.

" " $R_2 = \frac{1}{2}w + \frac{1}{2}W + w_1$.

Comp. in $A = \frac{w + W}{2} \times \frac{A}{H}$.

" " $B = (\frac{1}{2}w + \frac{1}{2}W + w_1 + W_1) \times \frac{B}{H}$.

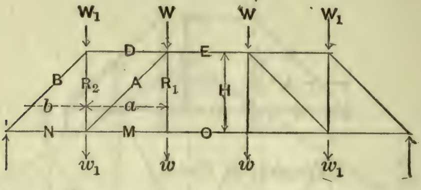
Tension in $N = (\frac{1}{2}w + \frac{1}{2}W + w_1 + W_1) \times \frac{b}{H}$.

" " M = tension $N + \left[\frac{w + W}{2} \times \frac{a}{H}\right]$

Compression in D = tension in N.

*Meaning length of B divided by height H, both in the same unit of measurement.

TABLE XVII. — STRESSES IN FIVE-PANEL HOWE TRUSS. TRUSS SYMMETRICAL AND SYMMETRICALLY LOADED.



Tension in $R_1 = w$.

" " $R_2 = w + w_1 + W$.

Compression in $A = (w+W) \times \frac{A}{H}$.

" " $B = (w + w_1 + W + W_1) \times \frac{B}{H}$.

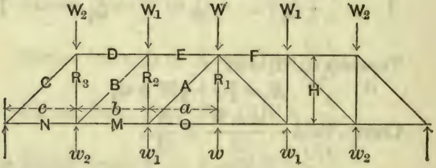
Tension in $N = (w + w_1 + W + W_1) \times \frac{b}{H}$.

" " M and O = tension in $N + \left[(w+W) \times \frac{a}{H}.\right]$

Compresion in D = tension in N.

" " E = tension in M.

TABLE XVIII. — STRESSES IN SIX-PANEL HOWE TRUSS. TRUSS SYMMETRICAL AND SYMMETRICALLY LOADED.



Tension in $R_1 = w$.

" " $R_2 = \frac{1}{2}w + w_1 + \frac{1}{2}W$.

" " $R_3 = \frac{1}{2}w + w_1 + w_2 + \frac{1}{2}W + W_1$.

Compression in $A = (\frac{1}{2}w + \frac{1}{2}W) \times \frac{A}{H}$.

" " $B = (\frac{1}{2}w + w_1 + \frac{1}{2}W + W_1) \times \frac{B}{H}$.

" " $C = (\frac{1}{2}w + w_1 + w_2 + \frac{1}{2}W + W_1 + W_2) \times \frac{C}{H}$.

Tension in $N = (\frac{1}{2}w + w_1 + w_2 + \frac{1}{2}W + W_1 + W_2) \times \frac{c}{H}$.

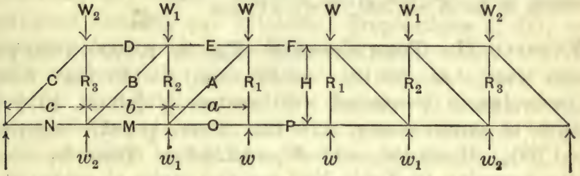
" " M = tension in $N + \left[(\frac{1}{2}w + w_1 + \frac{1}{2}W + W_1) \times \frac{b}{H}\cdot\right]$

" " O = tension in $M + \left[(\frac{1}{2}w + \frac{1}{2}W) \times \frac{a}{H}\cdot\right]$

Compression in D = tension in N.

" " E = tension in M.

TABLE XIX.—STRESSES IN SEVEN-PANEL HOWE TRUSS. TRUSS SYMMETRICAL AND SYMMETRICALLY LOADED.



Tension in $R_1 = w$.

" " $R_2 = w + w_1 + W$.

" " $R_3 = w + w_1 + w_2 + W + W_1$.

Compression in $A = (w + W) \times \frac{A}{H}$.

" " $B = (w + w_1 + W + W_1) \times \frac{B}{H}$.

" " $C = (w + w_1 + w_2 + W + W_1 + W_2) \times \frac{C}{H}$.

Tension in $N = (w + w_1 + w_2 + W + W_1 + W_2) \times \frac{c}{H}$.

" " M = tension in $N + \left[(w + w_1 + W + W_1) \times \frac{b}{H}\cdot\right]$

" " O and P = tension in $M + \left[(w + W) \times \frac{a}{H}\cdot\right]$

Compression in D = tension in N.

" " E = tension in M.

" " F = tension in O.

Examples Showing Application of Tables.

EXAMPLE I.—Simple fan truss of 36 ft. span. Distance between centres of trusses 12 ft. Height of truss 9 ft. or $\frac{S}{H}=4$. Total load per square foot of roof 40 lbs.:

Length of rafter 20 ft., nearly.

Panel load $W=\frac{20}{3}\times 12'\times 40=3{,}200$ lbs.

Then from Table X:

Stress in bottom of rafter $=3{,}200\times 5.59=17{,}888$ lbs.

Stress in ends of main tie $(F)=3{,}200\times 5.00=16{,}000$ lbs.

Stress in centre of main tie $=3{,}200\times 3.00=9{,}600$ lbs.

Stress in braces D and $E=3{,}200\times 1.08=3{,}456$ lbs.

Stress in tie $K=3{,}200\times 2=6{,}400$ lbs.

EXAMPLE II.—Truss shown in Fig. 5, p. 956 (four-panel Howe truss). $H=68$ ins., $a=108$ ins., $b=116$ ins. Length of inner braces (measured from centres of joints), $127\frac{1}{2}$ ins. Length of outer braces, 134 ins. From p. 957, $w=1{,}640$; $w_1=1{,}700$; $W=6{,}430$, and $W_1=6{,}320$ lbs. Then by means of the formulas in Table XVI we find stress in centre rod = 1,640 lbs.

Stress in outer rods $=\frac{1640}{2}+\frac{6430}{2}+1{,}700=5{,}735$ lbs.

Stress in inner braces $=\frac{1640+6430}{2}\times\frac{127\frac{1}{2}}{68}=7{,}536$ lbs.

Stress in outer braces $=\left(\frac{1640}{2}+\frac{6430}{2}+1{,}700+6{,}320\right)\times\frac{134}{68}=23{,}755$ lbs.

Tension in end panels of tie-beam

$$=\left(\frac{1640}{2}+\frac{6430}{2}+1{,}700+6{,}320\right)\times\frac{108}{68}=19{,}028 \text{ lbs.}$$

Compression in top chord $=19{,}028$ lbs.

The Graphic Method of Determining the Stresses in Roof Trusses.

The "Graphic Method" is the simplest and in most cases the quickest method (provided the tools are at hand) of determining the stresses in a roof truss, and, besides these, it has the additional advantages that it can be used for any true truss and for any arrangement of loads. There is also less

chance of making a mistake in the graphic method than by numerical computations, as an error in the graphical analysis almost always becomes manifest.

Stress diagrams can be very quickly drawn when once the principle is understood, and without the aid of books or tables. For the forms of trusses in common use, the method of drawing the stress diagrams is quite simple, and a careful study of the following examples supplemented by a little practice in drawing the diagrams should enable any architect, draughtsman, or builder to grasp the principle.

Principles upon which the Graphic Method is based.—To thoroughly understand this method, a knowledge of the composition and resolution of forces as explained in Chapter VI is essential, and before studying this subject the student should read carefully pp. 231–233. Propositions I, III, and IV on those pages form the basis of graphic statics. In the graphic method all forces, including the loads, are represented by straight lines, and the direction of the force must be constantly kept in mind, and often it is of assistance to indicate the direction by an arrow-head as explained on p. 232. The direction in which a force acts also tells whether it is a pushing or pulling force, or whether the member in which the force or stress acts is in compression or tension. This is more fully explained on the following pages, and also in connection with several of the stress diagrams.

Forces which Act In and On a Truss.—Every stress diagram represents three sets of forces, viz., the external loads, the supporting forces, and the stresses in the truss members.

Supporting Forces.—For a truss to stand in place, the supports of the truss, taken together, must be capable of offering a reaction equal to the total load on the truss, including the weight of the truss itself. Each of these reactions must be represented as one of the forces acting on the truss when drawing the stress diagram; they will be hereinafter referred to as the supporting forces.

When the loads are symmetrical on each side of the centre of the span, the supporting forces will be equal, and each will be equal to one-half of the total load on the truss. When the loads are not symmetrical about the centre, either as regards point of application or magnitude, the supporting forces will be unequal and in most cases must be determined before the stress diagram can be drawn. The supporting forces for

unsymmetrically loaded trusses may be computed by the method explained on pp. 275–277.

Application of Graphic Statics to Simple Triangular Frames Having but One External Load.

The simple triangular frame is much used in building construction, and many forms of roof trusses are simple combinations of such triangles. It is therefore worth while to show how easily the above principles may be used to determine the stresses in such a frame.

In Diagram 1, Fig. 6, we have two struts abutting at the top

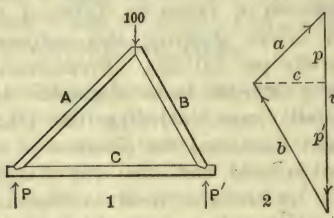


Fig. 6

and held both vertically and horizontally at the bottom by a tie-beam. The vertical component of the thrust in the strut, however, merely passes through the tie-beam and is resisted by the support below.

We will assume that a load of 100 lbs. is applied at the apex and disregard the weight of the frame itself. Now, if at 2 we draw a vertical line w, 1 in. long (scale of 100 lbs. to the inch) and from the upper end draw a line parallel to A, and from the lower end a line parallel to B, until the lines intersect, then the length of the line a, measured to the scale of 100 lbs. to the inch, will give the compressive force in A and the line b the compressive force in B. Further, if from the intersection of a and b we draw a horizontal line (parallel with the tie-beam) intersecting the line w, then the length of the line c will give the horizontal stress in the tie-beam produced by the load of 100 lbs. Moreover, the line c will divide the line w in the proportion of the reactions of the supports. Thus the portion p will be the amount of reaction at P and p' the reaction of P'.

All of these conditions remain true whatever the inclination of the struts, whether equal or unequal, and also if the tie-

beam is inclined, provided that the lines a, b, and c are drawn parallel to the pieces A, B, and C of the frame.

Moreover, the stresses will be proportional to the load at the apex. Thus for 200 lbs. the stress in each part will be just twice what it is for 100 lbs.

In Fig. 7 we have a load supported by two ties instead of two struts, the effect on the rod being the same as if the load were suspended from the bottom.

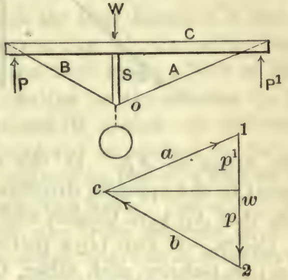


Fig. 7

If we let the vertical line 1–2 represent the load W, then the lines a and b, drawn parallel to A and B respectively, will represent the stress or tension in the two parts of the rod, and a horizontal line drawn from c to the vertical line will represent the compression in the strut C, and p' will be the reaction at P' and p the reaction at P. The stress in the post S will be equal to W.

The direction in which the forces act are determined as follows: Dead loads always act downward (hence are represented by vertical lines), and consequently the arrow-head on line 1–2 must point down.

The forces in b and a must also act in the direction of the arrow-heads, i.e., around the figure, in order to preserve equilibrium. Now the lines a, b, and w represent the three forces acting at o, and we see that the arrow-heads in a and b point away from the joint, hence these pieces are in tension. The arrow-head on w points towards the joint, hence S is in compression.

In Fig. 8 we have a crane supporting a load W.

If we draw the vertical line dc to represent the load, and from the lower end, or c, a line parallel to AC, and from the upper end a line parallel to BC, the two lines intersecting at e, then a will represent the stress in AC and b the stress in BC.

Considering the forces as acting at C, the direction in which the forces act are as indicated by the arrow-heads.

The arrow-head on a points away from C, hence AC is in tension; the head on b points towards C, hence BC is in compression.

We will next consider the forces which act at the point A. Of these three forces we have the force in AC represented

by the line a. If from e we draw a line parallel to AE, intersecting w at o, then eo will represent the stress in AE, and oc

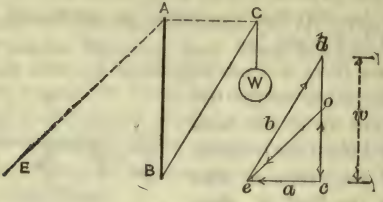


Fig. 8

the stress in AB. In the triangle eco, the arrow-heads will point in the opposite direction from what they do in ecd, showing that AE is in tension and AB in compression.

[NOTE.—If two boys pull on the two ends of a rope so as to just balance each other, the stress in the rope will be just the force with which one boy pulls, and each end of the rope will pull away from the boy holding it by the same force that he exerts. Thus if each boy exerts a force of 100 lbs.,

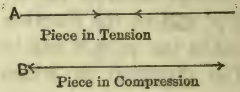


Fig. 9

then the stress in the rope will be 100 lbs., and each end of the rope will be pulling away with a force of 100 lbs. If the boys were pushing against the two ends of a piece of timber with a force of 100 lbs., then the timber would push against each boy with a force of 100 lbs., although the entire stress in the timber would be but 100 lbs.

Consequently a stress line with arrow-heads pointing *toward* each other, as at A, Fig. 9, denotes *tension*, and a stress line with arrow-heads pointing in *opposite directions*, as at B, denotes *compression*. In other words, *the stress in any member of a truss acts in opposite directions at the two ends of the piece.* This is an important truth to remember in drawing stress diagrams.]

Stress Diagrams for Vertical Loads.

Trusses Symmetrically Loaded.—Before the stress diagram for a truss can be drawn, it is necessary to make a skeleton drawing of the truss, representing the centre lines of the members as explained on p. 957. This diagram (which will

be hereinafter designated as the "Truss Diagram") should be drawn on the same sheet of paper as the stress diagram for convenience in drawing the latter. The truss diagram should also have all of the loads which come on the truss indicated by arrows and figures as in Fig. 10, which is the truss diagram for the truss represented by Fig. 1, and for which the loads were computed on p. 953.

Combining the Ceiling Loads with the Roof Loads.—It should be noticed that in the truss diagram, Fig. 10, the ceiling loads found on p. 953 are added to the roof loads. This is done to simplify the stress diagram. As far as the stresses in the struts and tie-beams are concerned it makes no difference whether the ceiling loads are considered as applied at the top or bottom of the truss, but the stresses in the rods will be increased by just the amount of the ceiling load. The +2070 lbs. opposite the centre rod is put on the truss diagram as a reminder to add this load to the stress afterwards determined. The rods from 2 to 6 and 4 to 8, Fig. 10, receive no stress from the roof loads and are therefore omitted or dotted in the truss diagram, the latter being lettered as though there were no rods there. The stress on these rods is simply that of the ceiling load at 6 and 8.

Whenever the ceiling loads are carried directly to the top by vertical rods or ties it is much simpler to add them to the roof loads, as above described, but when the ties are *not vertical* the ceiling loads must be indicated at their point of application.

Supporting Forces.—The supporting forces should also be indicated on the truss diagram as in Fig. 10. These forces are computed as explained on p. 967.

Lettering the Truss Diagram.—After the truss diagram is drawn, it should be lettered after a particular method known as "Bow's Notation," which enables a ready comparison of the truss and stress diagrams and also aids the student in drawing the stress diagram and in tracing the stresses. The essential principle of this method of lettering is to letter the *space* each side of every force or piece of the truss so that on the truss diagram a piece or force is denoted by the letters *on each side* of it. When the stress diagram is drawn it will be found that the same letters come at the end of the corresponding lines.

Fig. 10 shows the truss diagram of the truss represented in Fig. 1, properly drawn, lettered, and figured, ready for drawing

the stress diagram. The supporting force at the left is *AO*, the bottom of the main rafter *AE*, the left portion of the tie-beam *EO*, etc. The loads acting at joints 2, 3, and 4 are designated as *AB*, *BC*, and *CD* respectively. It makes no particular difference what letters are used, except that it is better to letter the outside spaces consecutively and then the inside spaces.

Stress Diagram.—The stress diagram is drawn by taking the forces acting on the joints in consecutive order, commencing at one of the supports. The author considers it more natural and convenient to start with the support at the left, or at joint 1.

[NOTE.—In actual computations it is not necessary to number the joints, but in order to refer to them in the description it is necessary to number them in the illustrations.]

Commencing at joint 1, then, the first step of the stress diagram is to draw a vertical line to a scale of pounds to the inch to represent the supporting force *OA*. This line is the line *oa*, Fig. 10 *A*, which is here drawn to the scale of 16,000 lbs. to the inch.* It is best to use a scale as large as convenient and not have the diagram too large. An engineer's scale, one divided to 10ths, 20ths, 30ths, etc., of an inch, will be found most convenient for these drawings.

The small letter *o* should be placed at the bottom of the line and the letter *a* at the top. Next from *a* draw a line parallel to the rafter *AE* and from *o* a line parallel to the tie-beam *OE*. The two lines meet at *e*, and *ae* represents the stress in *AE* and *oe* the stress in *OE*. As the supporting force acts up, the arrowhead will be at the top of *oa*, and the others must follow in rotation, showing that *ae* acts *toward* the joint and the piece is in compression, and *eo* acts *from* the joint and the piece is in tension.

We next consider the stresses at joint 2. Commencing at the bottom of the joint and going around to the left the first stress that we know is the stress in *ae*, which we have just determined. As this stress acted downward at 1, it will act *upward* at 2, as the stresses in the two ends of a strut or tie act in opposite directions, as explained on p. 970. The stress *ae* we determined in Diagram 10A, and for convenience in explanation we will consider it redrawn in Fig. 10B. The next force

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* The original of this drawing was at a scale of 8,000 pounds per inch, the drawing being reduced one half in making the cut.

is the load $AB = 9{,}000$ lbs., which we measure to our scale from *a* downward (as the loads acts down), which gives us the point *b*. The stresses in *BF* and *EF* we do not know, so from *b* (Fig. 10, *B*) we draw a line parallel to *BF*, and from our starting-point, *e*, a line parallel to *EF*, and we obtain the lines *bf* and *fe*, which represent the stresses in *BF* and *FE* respectively. The arrow-heads should follow as indicated, all of the parts being

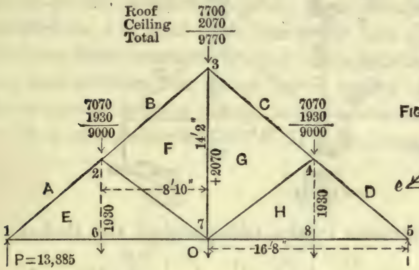


FIG. 10. TRUSS DIAGRAM

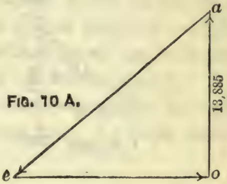


FIG. 10 A.

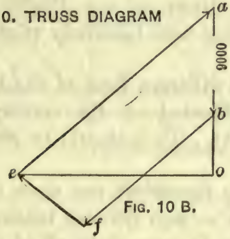


FIG. 10 B.

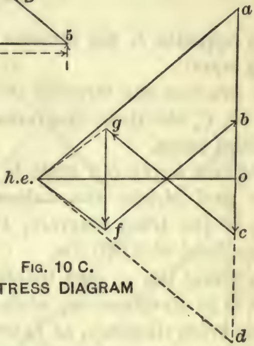


FIG. 10 C.
STRESS DIAGRAM

in compression. At joint 7 we now know the stresses in *OE* and *EF*, leaving three unknown forces and as we can only determine two forces we must go to joint 3, where there are but two unknown forces. The first force which we know at 3 is the stress *fb*, which now acts up; we then have the load *BC* of 9,770 lbs., which we measure off from *b* (Fig. 10, *C*) to our scale, which gives us the point *c*. From *c* we draw a line parallel to *CG*, and from *f* a line parallel to *FG*, and we have the lines *cg* and *fg*, which represent the stresses in *CG* and *FG* respectively. The arrow-heads follow as shown, *gf* acting downward or from the joint 3, and hence indicating tension. As the truss is symmetrical, we now have determined the stresses in all of

the parts, but we can continue the process if we wish, when we will obtain the stresses shown by the dotted lines, the diagram being symmetrical about the line *eo*. The letter *h* will

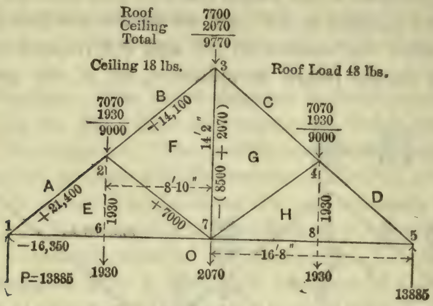


Fig. 11

come opposite *e*, the stresses in the two parts of the tie-beam being equal.

In practice the stresses are all drawn on one diagram, as in Fig. 10, *C*, the three diagrams being used here only to show the different steps.

We now apply our scale to the different lines of the last diagram and obtain the values indicated on the corresponding lines of the truss diagram, Fig. 11. To *fg* should be added the ceiling load of 2,070 lbs.

By using the + sign before the figures we can show that the piece is in compression, while the − sign denotes tension.*

The truss diagram, as figured in 11, now gives all of the data required to determine the size of the parts, and when these are determined they should also be marked on the diagram so as to have a complete record of the whole computation.

In practice diagrams Figs. 10 and 11 would be combined in one drawing, they being shown separately merely to indicate the progressive steps in lettering and figuring.

Example 2 (Fig. 12).—The upper diagram in Fig. 12 represents the centre lines of the truss shown by Fig. 3, and the loads indicated are those found in Example 2, p. 953. The counter braces in the centre panel are indicated by dotted lines

* There is no uniformity in the use of these signs by different writers, some using + to denote tension; hence the notation should be looked up in each work.

in the truss diagram, because under a symmetrical load there is no stress in these members, and hence they cannot be represented in the stress diagram.

As in the foregoing example, the ceiling loads are added to the roof loads and treated as one load.

As the truss is symmetrically loaded each supporting force will be equal to one-half of the total load, or 18,500 lbs.

To draw the stress diagram, first draw a vertical line *oa* equal to the supporting force, then from the upper end a line parallel to *AE*, and from the lower end a line parallel to *OE*, the two lines intersecting at *e*. We will thus have the triangle *oae* representing the three forces acting at the joint 1. As the supporting force *always acts up*, the arrow-head on *oa* will be at *a*, on *ae* at *e*, and on *eo* at *o*, showing that *AE* is in com-

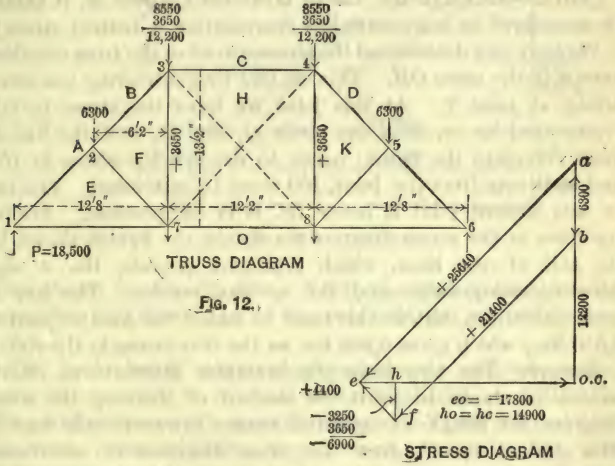


FIG. 12.

pression and *EO* in tension. Next find the stresses acting at joint 2. We already have the stress in *AE*, represented by the line *ea*, and as the stress will act in the opposite direction at joint 2 from what it does at joint 1, it will now act up. The next force is the load of 6,300 lbs., which must act downward from *a*. We obtain the point *b* by measuring from *a* a distance =to 6,300 lbs. (at the same scale as was used in drawing *oa*). There now remain two stresses to be found, viz., the stresses in *BF* and *FE*. From the point of beginning *e*, draw a line parallel

to EF, and from b a line parallel to BF, the two lines intersecting at f. Then the figure *eabf* will represent the four forces acting at joint 2, and the stresses will act in the directions indicated by the letters, or all act toward the joint. We may next obtain the forces acting at either joints 3 or 7, as there are but two unknown forces at either joint.

Considering the forces acting at joint 3, we already have the force in FB, represented by the line *fb*, which acts up then the load of 12,200 lbs., which takes us to o, where we also put the letter c to conform with the lettering on the truss diagram. From c draw a horizontal line (parallel to CH), and from the point of beginning, f, a line parallel to FH, the two lines intersecting at h. *ch* will represent the stress in CH and *fh* the stress is FH.

[NOTE.—Although the line *ch* lays over the line *oe*, it should be considered as a separate line representing a distinct stress.]

We have now determined the stresses in all of the truss members except in the piece OH. This we find by considering the forces acting at joint 7. At this joint we have the stress in OE, represented by *oe*, next the stress *ef*, also *fh*, and the line *ho* must complete the figure; hence *ho* denotes the stress in HO, and as it acts from the joint, HO must be in tension. The line *ch* acts toward joint 3, hence HC is in compression. Scaling the lines in the stress diagram we obtain the figures shown by the side of the lines, which represent pounds, the + sign denoting compression and the − sign tension. The line *hf* scales 3,250 lbs., but to this must be added the load at joint 7, 3,650 lbs., which gives 6,900 lbs. as the true stress in the rod.

Remark.—The two foregoing examples illustrate as clearly as can be shown in print the method of drawing the stress diagram for simple symmetrical trusses symmetrically loaded. The student should draw the truss diagrams in accordance with the measurements given, but to a scale of not less than $\frac{1}{8}$ inch to the foot, and then draw the stress diagram, line by line, in accordance with the foregoing directions and compare the results obtained with those given in the figures. A variation of 100 or even 200 pounds may be expected, but a greater variation will indicate either that sufficient care has not been exercised in drawing the stress lines exactly parallel with the corresponding lines of the truss diagram or that an error has been made in drawing the truss diagram or in scaling the lines of the stress diagram.

After these two examples have been worked, a number of the following examples should be worked until the entire principle is fully understood.

EXAMPLE 3.—Fig. 13 represents the truss diagram of the

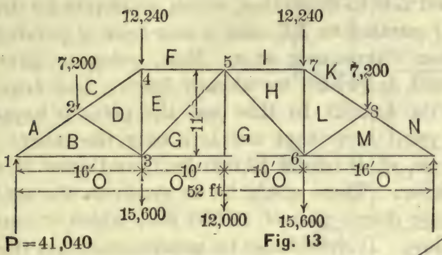


Fig. 13

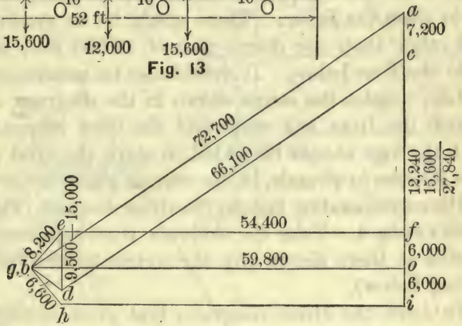


Fig. 13 A

truss shown by Fig. 13, Chap. XXV, the loads indicated being approximately those due to the roof and the suspended floor below.

The loads being symmetrically disposed, each supporting force will be equal to one-half of the total load, or 41,040 lbs. The counter braces *CC*, shown in Chap. XXV, are omitted from the truss diagram because they have no stress when the truss is uniformly loaded.

To draw the stress diagram, first draw the vertical line $oa = 41{,}040$ lbs. $= P$, then *ab* and *ob* parallel to *AB* and *OB* and representing the stresses acting at joint 1. At joint 2 we have the line *ba* representing the stress in *BA*, and from *a* measure down $ac = 7{,}200$ lbs., then draw *cd* and *bd*, *bacdb* representing the forces acting at joint 2.

At joint 3 we have three unknown forces, and as we cannot find three unknown forces in one polygon, we must go next to joint 4, where we already have *dc* and the load *CF*.

[NOTE.—In Fig. 13 the bottom loads are not shown added to the top loads, but they should be so added before drawing the stress diagram.]

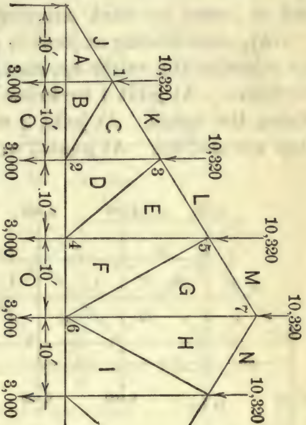
Measuring off the force $cf = 27{,}840$ lbs., we have only the stresses in FE and DE to determine, which we obtain by drawing a line from f parallel to FE, and a line from d parallel to DE, the two lines intersecting at e. Now, going to joint 3, we have *ob*, *bd*, and *de* (which we already know), and draw *eg* and *og* to close the figure. In this case the point g happens to come at the point b, so that one lays over the other. At joint 5 we have *ge*, *ef*, *fi* (equal 12,000 lbs.), and draw *ih* and *gh* to close the figure. There would be no strain on the central rod other than the direct pull of 12,000 lbs., which it carries from the floor below. It should also be remembered that the tie *de* has, besides the stress shown in the diagram, a direct pull of 15,600 lbs. from the weight of the floor suspended from it, so that the two should be added to show the total pull in the rod. The stresses in pounds, in the various pieces are given in numbers on the corresponding lines in the stress diagram (Fig. 13 A).

EXAMPLE 4.—Take the skeleton truss represented in Fig. 14, loaded as there shown (by the weight of the roof above and a ceiling below).

To draw the stress diagram, first draw the supporting force *oj* (Fig. 14 A) $= 46{,}620$ lbs. Then for convenience measure off from j, *jk*, equal to the sum of the weights at joints o and 1 (13,320 lbs.), $kl = 13{,}320$, $lm = 13{,}320$, and $mn = 13{,}320$. Then draw the lines *ja* and *oa* and we have the stresses at the support. At joint 1 we know *aj* and *jk*, and draw *kc* and *ac* to close the figure. There will be no line in the stress diagram corresponding to AB, for there is no stress in that tie excepting the direct pull of 3,000 lbs. At joint 2 we have *oa* and *ac*, and draw *cd* and *od* to close the figure. At joint 3 we have *dc*, *ck*, and *kl*, and draw *le* and *de*. At joint 4 we already have *od* and *de*, and find *ef* and *of* by drawing lines from e and o parallel to the respective pieces in Fig. 14. At joint 5 we have *fe*, *el*, and *lm*, and draw *mg* and *fg*. We must next go to joint 7; for at joint 6 we would have three stresses to find, and by the graphic method we can find only two at a time. At joint 7 we have *gm* and *mn* (13,320 lbs.), and draw *nh* and *gh* to close the figure. This completes the stresses in all the pieces for one-half of the truss, and of course the stresses for each half are the same.

EXAMPLE 5 (Eight panel Howe Truss). — For the next

example we will take a Howe truss whose centre lines give the diagram shown by Fig. 15. This truss is for a span of sixty-four feet, and supports a flat roof and plaster ceiling below



P=46,620

Fig. 14

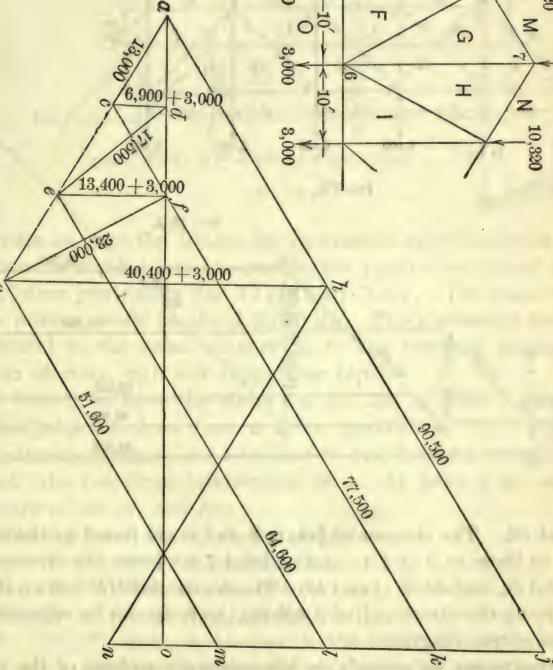


Fig. 14 A

the tie-beam, and also a gallery below *on each side.* The loads at the different joints would be about as indicated in Fig. 15. To draw the stress diagram (Fig. 15 A) lay off the loads on a

vertical line, commencing first with the loads nearest the support. Thus *ab* equals load at joints 1 and 2, *bc* equals load at joints 3 and 4, *cd* equals load at joints 5 and 6, and *do* and *oe* each equals one-half of loads at 7 and 8, because one-half of this load is borne by each support.

Next, commencing at joint *o*, we have the supporting force *oa*, the stress in the rafter *ap*, and the stress in the tie *po*, closing the figure. At joint 1 we know *pa* and *ab*, and draw *bn* and *pn*, closing the figure. At joint 2 we know *op* and *pn* already, and draw *nm* and *om*. At joint 3 we know *mn*, *nb*, and *bc*, and draw

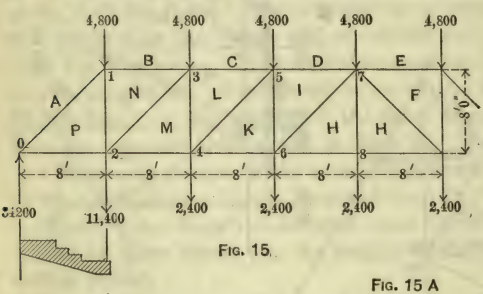


FIG. 15.

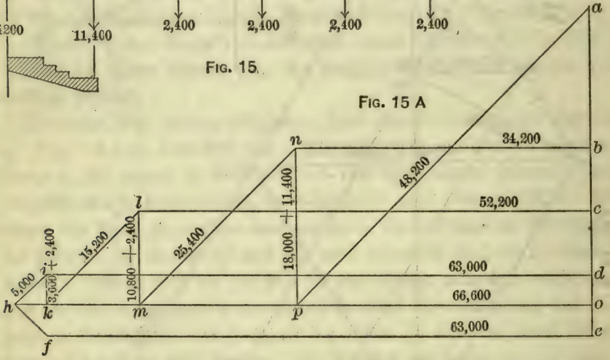


FIG. 15 A

cl and *ml*. The stresses at joints 5 and 6 are found in the same way as those at 3 and 4; and at joint 7 we know the stresses *hi*, *id*, and *de*, and draw *ef* and *hf*. The centre rod *HH* has no stress excepting the direct pull of 2,400 lbs., so it cannot be represented in the stress diagram.

The stresses, in pounds, in the various members of the truss are given on the stress diagram. These stresses can be checked by means of the formulas given for a six-panel truss (Table XVIII) carried one step further.

EXAMPLE 6.—*Howe Truss loaded at alternate joints* (Fig. 16). This example has been selected to show how to proceed when there is no load at one or more of the joints. Fig. 16 represents the centre lines of a truss of 50 ft. span and only 5 ft. in height.

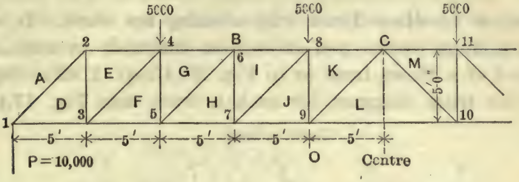
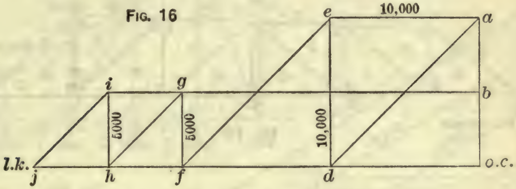


FIG. 16



$bg = 20{,}000$; $bi = 25{,}000$; $oj = ck = 30{,}000$

FIG. 16 A

In order to give the braces an inclination approximating 45° the truss is divided into ten panels, but purlins are placed over every other joint, as in Fig. 19 of Chap. XXV. The loads from these purlins would be about 5,000 lbs. The stresses at joint 1 are found in the same manner as in the previous examples, always starting with the supporting force.

At joint 2 we have the stress line *da*, and as there is no load at this joint we draw from *a* a line parallel to *AE* (*A* covers the entire space from joint 1 to joint 4), and from *d* a line parallel to *DE* the two lines intersecting at *e*. At joint 3 the stress lines are *od*, *de* *ef*, and *fo*:

Stresses at joint 4, *fe*, *ea*, *ab*, *bg*, and *gf*.
" " " 5, *of*, *fg*, *gh*, and *ho*.
" " " 6, *hg*, *gb*, *bi*, and *ih*.
" " " 7, *oh*, *hi*, *ij*, and *jo*
" " " 8, *ji*, *ib*, *bc*, and *ck*, the latter line bringing us to the point of beginning, showing that there is no stress in *kj*.

At joint 9 the only stresses are *oj* and *lo*, for as there is no stress in *JK* there can be none in *KL*.

There would also be no stress in the centre rod. Although these members have no stress it is advisable to insert them in the truss to stiffen the top and bottom chords, but they can be made very light, say ½ inch for the rods and 3×6 for the braces.

EXAMPLE 7.—*Howe Truss with slanting top chord.* In order to give a slope to the roof it is often desirable to incline the top chord of a Howe truss as in Fig. 20, Chap. XXV. Fig. 17 shows the truss diagram for such a truss, and Fig. 17*A* the

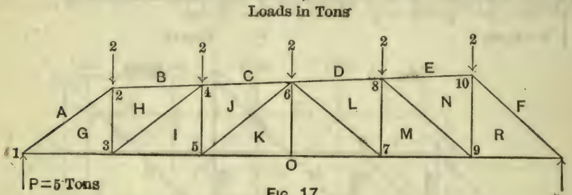


FIG. 17

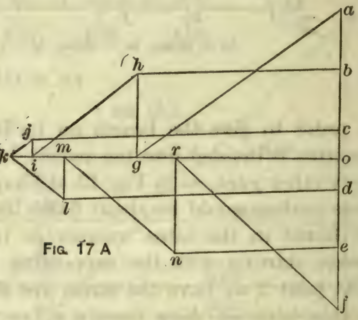


FIG. 17 A

stress diagram. The latter is drawn in the same way as the stress diagram in example 5, but because the top chord is not level, the stress diagram will not be symmetrical. When the stress diagram is not symmetrical it is necessary to complete the entire diagram, so as to show the stress in every member of the truss; the stress lines for joint 9 are *om*, *mn*, *nr*, and *ro*. This leaves only the line *rf* to complete the diagram, and if the diagram has been correctly drawn, a line joining *r* and *f* will be exactly parallel to *RF*. There will be no stress in the centre rod.

EXAMPLE 8.—*Truss with inclined ties* (Fig. 18). This truss has the same dimensions as the truss shown by Fig. 14, but the

diagonals incline in the opposite direction and are in tension, and the verticals, except at the centre, are in compression.

This form of truss is sometimes used in wooden construction to avoid the long centre braces which occur in Fig. 14. Long ties being, as a rule, more economical than long struts.

For this truss we *cannot add* the ceiling loads to the roof loads, because the effect on the ties is greater than the amount of the loads.

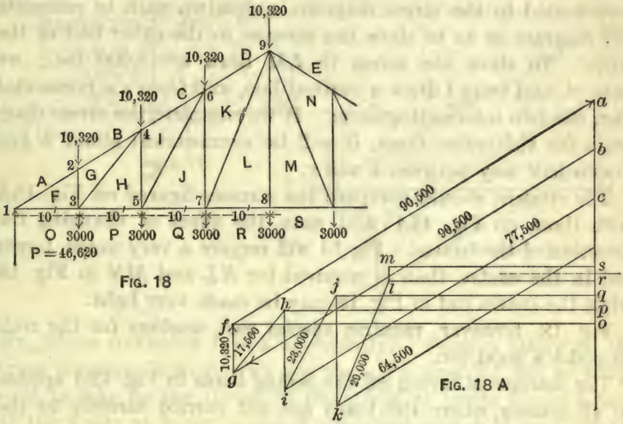


Fig. 18

Fig. 18 A

To draw the stress diagram (Fig. 18*A*), first draw *oa* = to the supporting force (46,620 lbs.) and from *a* and *o* draw lines parallel to *AF* and *OF*, intersecting at *f*. The triangle *oaf* is the same as in Fig. 14*A*, showing that the maximum stresses in rafter and tie-beam are the same as in the truss, Fig. 14. Having found the forces at joint 1, we proceed to joint 2, where we have *fa* acting up, *ab* = 10,320 lbs., and draw *bg* and *fg*. The arrow-head on *gf* points up, or towards the joint, showing that *FG* is in compression. Next go to joint 3. The first force which we know at this joint is the load of 3,000 lbs. As weights must always be represented by a vertical line acting down, and as the bottom of the line in this case must be at *o*, we measure *upwards* from *o* 3,000 lbs. and mark the point thus obtained *p*. Our polygon of forces for joint 3, then, commences at *p*, and we have *po*, *of*, and *fg*. Then from *p* draw a line parallel to *PH*, and from *g* a line parallel to *GH*, the two intersecting at *h*. Then *po*, *of*, *fg*, *gh*, and *hp* represent the

forces acting at joint 3. *gh* and *hp* both act from the joint, and hence are in tension.

The stresses at joint 4 are *hg*, *gb*, *bc*, *ci*, and *ih*.
" " " " 5 are *qp*, *ph*, *hi*, *ij*, and *jq*.
" " " " 6 are *ji*, *ic*, *cd*, *dk*, and *kj*.
" " " " 7 are *rq*, *qj*, *jk*, *kl*, and *lr*.

The stress in *LM* will be only that produced by the load which it directly supports, viz., 3,000 lbs., and it need not be represented in the stress diagram, unless we wish to complete the diagram so as to show the stresses in the other half of the truss. To show the stress in *LM* draw $sr = 3{,}000$ lbs.; we have *rl*, and from *l* draw a vertical line, and from *s* a horizontal line, the two intersecting at *m*. If we complete the stress diagram for the entire truss, it will be symmetrical about a line drawn half way between *s* and *r*.

The student should compare the stresses figured on Fig. 18*A* with those on Fig. 14*A* and note the effect of changing the direction of the braces. Fig 14 will require a very much larger rod in the centre than is required for *KL* and *MN* in Fig. 18 while the centre rod in Fig. 18 may be made very light.

Fig. 18, however, requires special cast washers for the rods to make a good job.

The manner of laying off the ceiling loads in Fig. 18*A* applies to all trusses where the loads are not carried directly to the top by means of vertical rods or ties.

EXAMPLE 9.—*Simple Fan Truss* (Fig. 19). In Fig. 19 we have the skeleton of a simple fan truss with inclination of 30° and the rafters divided into three equal panels, so that the loads are all alike.

The stress diagram is drawn on the same principle as those previously explained and involves no unusual points. As the loads are all alike, the stresses in this truss may be readily figured by means of Table X, and the student should compare the stresses thus obtained with those obtained by scaling the stress diagram.

EXAMPLE 10.—*Cambered Fink Truss* (Fig. 20). Inclination of rafters 30°. Distances between trusses 20 ft Loads are for slate roof on boards or angle-iron purlins. Commence the stress diagram by drawing a vertical line equal to the supporting force *P*, or 56,350 lbs., and lettering the bottom *o* and the top *a*, as these are the letters on each side of the supporting force at joint 0. Draw *an* and *on* parallel to *AN* and *NO*. At joint 1

we have *na* acting up; measure off ab=16,100 lbs., draw *bm* and *nm* parallel to *BM* and *NM*. At joint 2 we have *on* and *nm*, and draw *ml* parallel to *ML*, the stress polygon being *on*, *nm*, *ml*, and *lo*.

At joint 3 we meet a condition which we have not found in any of the preceding examples, and which is peculiar to this truss,

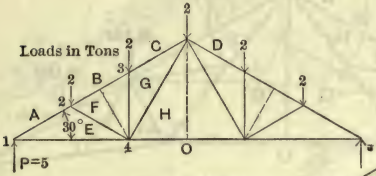
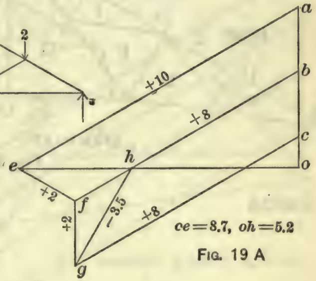


FIG. 19



ce=8.7, oh=5.2

FIG. 19 A

viz., three unknown forces apparently. From a study of the truss diagram, however, we see that *ML* and *KI* act as belly-rods to take up the thrust in the struts at joints 2 and 5, and as the loads at joints 1 and 6 are equal and *NM* and *IH* are of the same length, the stress in *KI* must be the same as the stress in *ML*, which we already know. This reduces the number of unknown forces at joint 3 to two.

The first force which we know at this joint is *lm*, the next *mb*, the next bc=16,100 lbs., and from *c* draw a line parallel to *CI*, and from *l*, the point of beginning, a line parallel to *LK*. Now between these two lines we must have a line, *ik*, parallel to *IK* and equal in length to *ml*; this line we obtain by means of the dividers and a parallel ruler, or triangle. If correctly drawn, the point *i* will be found in line with *nm*. The stress polygon for joint 3, then, is *lm*, *mb*, *bc*, *ci*, *ik*, and *kl*.

At joint 4 the stress lines are *ol*, *lk*, *kg*, and *go*.
" " 5 " " " " *gk*, *ki*, *ih*, and *hg*.
" " 6 " " " " *hi*, *ic*, *cd*, and *dh*.

If the stress diagram is accurately drawn a line from *d* parallel to the rafter will pass through the point *h*. The vertical tie *GG*,

Fig. 20, has no stress and its only duty is to prevent the horizontal tie from sagging.

EXAMPLE 11 (Fig. 21).—*Same Truss as in Fig. 20, carrying two additional loads.* Steel trusses of this shape are often

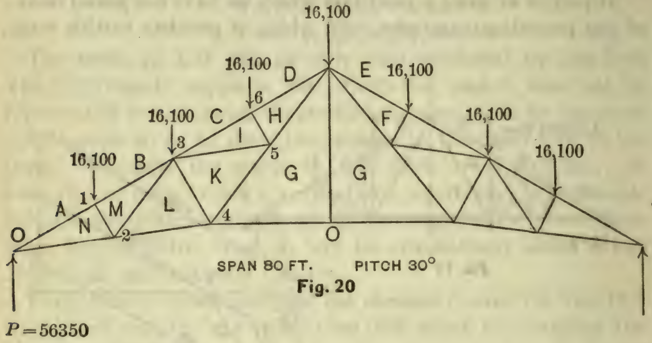


Fig. 20

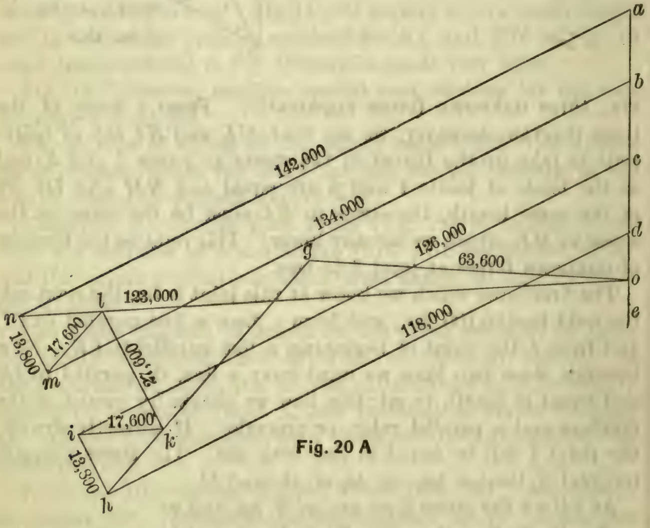


Fig. 20 A

required to support loads from below. In Fig. 21 we have two loads of 4 tons each, supported at joints 5 and 9, in addition to the roof loads.

The stress diagram is drawn in exactly the same way as Fig. 20*A*, except that at joint 5 the first known force is *RO*,

4 tons, and we lay off above o a distance equal to 4 tons, which gives us the point r. We then have at this joint ro, ol, lk, and draw kg and rg to close the figure.

It should be noticed that the stresses in NM, IH, ML, KI, and LK are the same as the stresses in the corresponding mem-

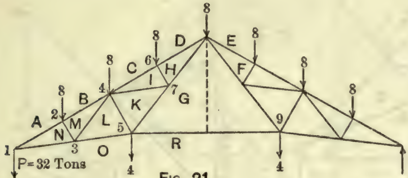


FIG. 21

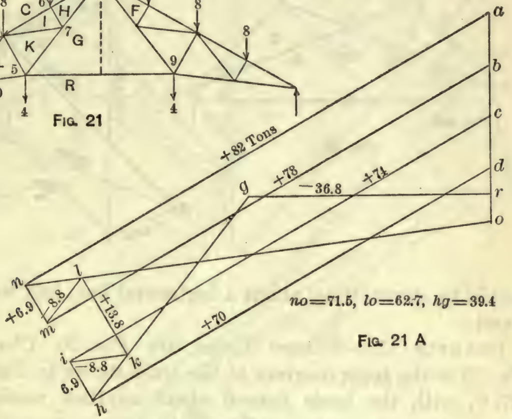


FIG. 21 A

bers of Fig. 20, as these are not affected by the ceiling load. All of the other stresses, however, are increased because of the increase in the supporting forces, the greatest increase, however, being in KG and HG.

EXAMPLE 12.—*Simple Scissors Truss* (Fig. 22). Fig. 22 is the truss diagram of the truss shown by Fig. 25, Chap. XXV, which is the simplest form of the scissors truss. The truss diagram is drawn by commencing with the line oa equal to the supporting force (9,600 lbs.) and drawing ad and od parallel to AD and OD. Next, at joint 2 we have da acting up; measure off ab = 5,800 lbs. and draw be and de parallel respectively to BE and DE. Next go to joint 3. Here we have eb acting up; measure off bc = 5,200 lbs. and draw cf and ef parallel to CF and EF. We now have the stresses in one half of the truss and the stresses in the other half will be the same. If we wish to complete the diagram, measure off above o, ro equal to the load at joint 4, 2,400 lbs., and draw rg and fg parallel to RG and FG. The stress polygon for joint 4 is ro, od, de, ef,

fg, and *gr*. For the stresses at joint 5, measure down from *c* a distance = 5,800 lbs. and draw a line parallel to the rafter which should pass through the point *g*. The completed figure

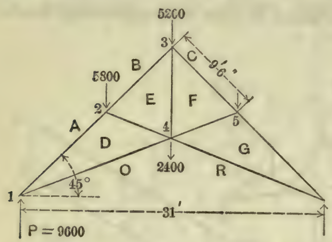


FIG. 22

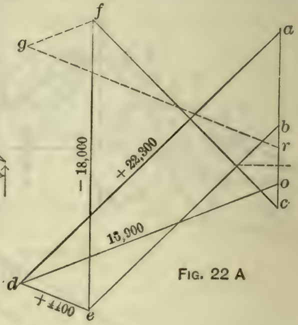


FIG. 22 A

should be symmetrical about a horizontal line, half way between *r* and *o*.

EXAMPLE 13.—*Scissors Truss like Fig*. 26, *Chapter XXV*. Fig. 23 is the truss diagram of the truss shown in Fig. 26, Chap. XXV, with the loads figured about as they would be for a slate roof and wooden ceiling with trusses spaced 12 ft. on centres.

Commence the stress diagram by drawing the line *oa* = to the supporting force at joint 1 (14,750 lbs.). The stress polygons for the different joints are as follows:

At joint 1: *oa*, *ae*, *eo*.
" " 2: *ea*, *ab*, *bf*, and *fe*.
" " 3: *oe*, *ef*, *fh*, and *ho*.
" " 4: *hf*, *fb*, *bk*, and *kh*.
" " 5: *ro* = 1,100 lbs., *oh*, *hk*, *kl*, and *lr*.
" " 6: *lk*, *kb*, *bc* = 5,400 lbs., *cm*, and *ml*.
" " 7: *mc*, *cd* = 5,700 lbs., *dn*, and *nm*.

The student should notice how much the stresses in the principal members of this truss exceed the supporting forces or loads, and particularly the great stress in the centre rod.

For this reason this is not an economical type of truss for spans exceeding 36 ft.

Example 14.—*Scissors Truss like Fig.* 4. Fig. 24 is the truss diagram of the truss shown by Fig. 4, p. 954, and for which the roof and ceiling loads are computed in Example 3, p. 954. The truss shown by Fig. 4 is built of planks spiked and bolted together, but the stresses would be found in pre-

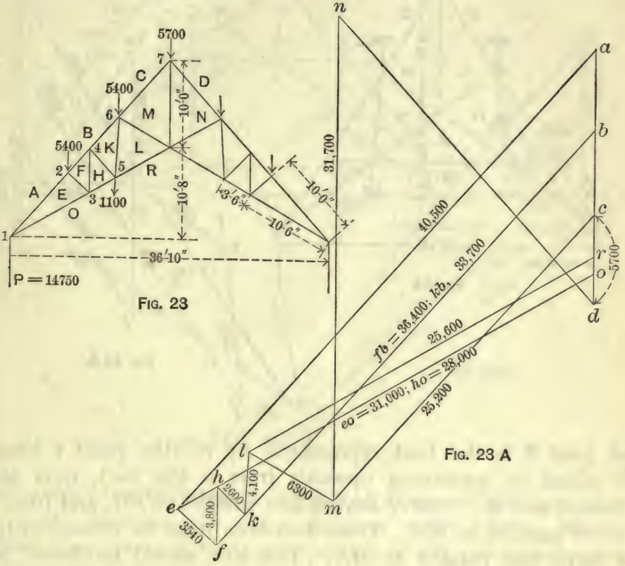


Fig. 23

Fig. 23 A

cisely the same way if the truss were made of heavy timbers and supported a greater roof area. It should be remembered that only the shape of the truss and the loads, including their point of application, affect the stress diagram.

The stresses at joints 1 and 2 are readily found commencing with $oa = P$. At joints 3 and 4, however, we have three unknown forces. We cannot obtain the stresses at joint 4 until we have drawn those acting at joint 3. The known forces at 3 are the load $OR = 430$ lbs. and the stresses acting in OE and EF, and the unknown forces those acting in FH, HK, and KR. Now both HK and KR are in tension and both serve to hold joint 3 from falling down and outwards. Either one (but not both) could be omitted, and the greater the stress in one the less will be the stress in the other. The only way in which we can complete the stress polygon for joint 3 is to fix the amount

of one of the unknown stresses arbitrarily. The most satisfactory analysis seems to be to make the stress in HK equal to that in KR. This is done as follows: The first known force

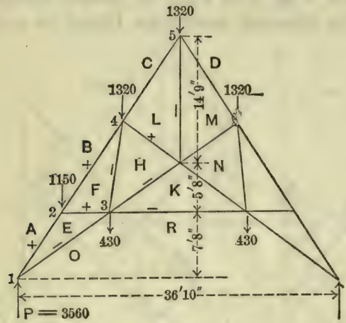


FIG. 24

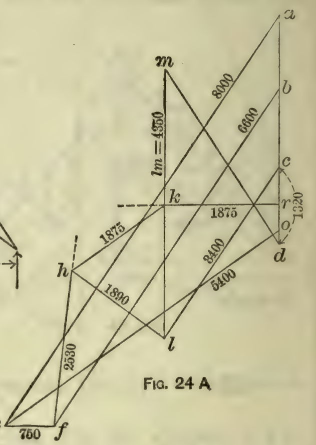


FIG. 24 A

at joint 3 is the load represented by *ro* (the point *r* being obtained by measuring upwards from *o* 430 lbs.), next the lines *oe* and *ef*. From *f* draw a line parallel to FH, and from *r* a line parallel to KR. These two lines must be connected by a third line parallel to HK. This line should be drawn so that its length will be equal to *kr*, which can be done by means of dividers. Lettering the ends of this line *h* and *k* we have the completed stress polygon for joint 3, viz., *ro*, *oe*, *ef*, *fh*, *hk*, and *kr*. Knowing the stress in FH, we have but two unknown forces at the joint 4, which are readily found.

The stress polygon for joint 5 is *lc*, *cd*, *dm*, and *ml*. Comparing this stress diagram with Fig. 23*A*, we see that the stress in the centre rod is much less in proportion to the loads in Fig. 24*A* than it is in Fig. 23*A*, this reduction being due to the horizontal tie RK. For light trusses built of planks spiked or bolted together the form of truss shown by Fig. 24 is preferable to that shown in Fig. 23.

EXAMPLE 15.—*Scissors Truss like Fig.* 30, *Chap. XXV.* Fig. 25 is the truss diagram of the truss shown by Fig. 30 of Chapter XXV. The line EF in Fig. 25 does not correspond

with the centre line of the strut in Fig. 30, because the inner end of the latter is dropped slightly on account of the detail

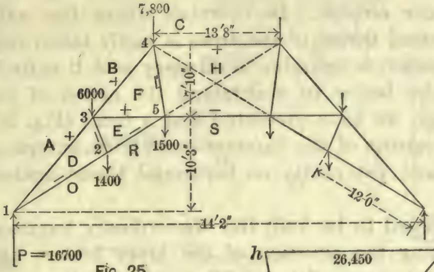


FIG. 25

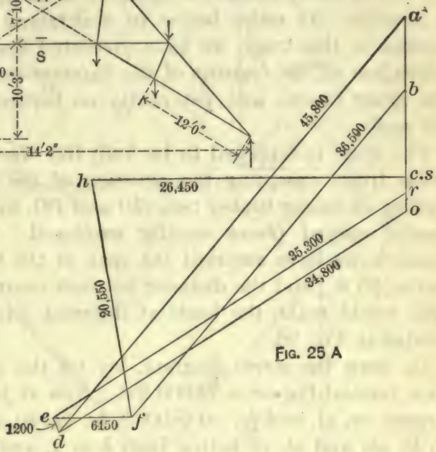


FIG. 25 A

of the joint, but in the truss diagram all lines must go from joint to joint, otherwise the stress diagram could not be drawn. There is no stress in the upper portion of the tie-beams under a symmetrical vertical load, hence they are shown by dotted lines in Fig. 25. There are no complications in drawing the stress diagram of this truss, therefore a detailed description is unnecessary.

The stress polygons for the different joints are as follows:

For joint 1: *oa, ad, do.*

" " 2: *ro, od, de, er.*

" " 3: *ed, da, ab, bf, fe.*

" " 4: *fb, bc, ch, hf.*

" " 5: *sr, re, ef, fh,* and *hs.*

hs and *hc* coincide, showing that the tension in *HS* is equal to the compression in *CH*. The plus and minus signs on Fig. 25 denote compression and tension respectively.

EXAMPLE 16.—*The Hammer-beam Truss.* As this truss is so frequently used by architects for supporting the roofs of churches and large halls, we have devoted considerable space to it.

As generally constructed, hammer-beam roof trusses exert a more or less horizontal pressure upon the walls supporting them, requiring that the walls shall be heavy and re-enforced by buttresses on the outside. In churches where the walls are low, this horizontal thrust of the truss is easily taken care of; but in many cases it is desirable to do away with it entirely if possible. In order better to understand the action of the stresses in this truss, we have presented first a truss (Fig. 26) which has all the features of the hammer-beam truss, excepting the lower braces, and yet exerts no horizontal thrust against the wall.

The truss is supposed to be built like the ordinary hammer-beam truss, excepting the omission of the lower braces, and putting in strong timber ties, *HO* and *PO*, in place of the ornamental curved pieces usually employed. In this particular example we have assumed the span of the truss as 60 ft., the rise as 35 ft., and the distance between centres of trusses 15 ft, This would make the loads at different joints about as is indicated in Fig. 26.

To draw the stress diagram, lay off the supporting force *P* on a vertical line *oa*,= 42,000 lbs. Now at joint 1 we have the stresses *oa*, *af*, and *fo*; at joint 2, *fa*, *ab*, *bg*, and *fg*; at joint 3, *of*, *fg*, *gh*, and *oh*, *oh* acting from *h* to *g*, and hence is a pulling stress. At joint 4 we have *hg*, *gb*, *bc*, *ci*, and *hi* to close the figure; *hi* is also in tension. At joint 5 we have *ic*, *cd*, *dk*, and *ik*. At the top joint 6 the stresses are *kd*, *de*, *el*, and *kl*, which completes the stress diagram for one-half of the truss, which, of course, is all that is needed. Examining, now, the diagram, we find that the stresses are in general much larger than would be the case if there were a horizontal tie across the truss; still, if we make the pieces large enough to withstand these stresses, the truss will be stable and exert no outward thrust on the walls.

Looking at Fig. 26 we see that *OF*, *H*, *P*, and *R* form a continuous tie, only it is pulled up in the centre in the form shown. In Fig. 26*A* we see that the stress in the tie-rod *KL* is very great, and this is because the rod has to hold up the inclined ties *HO* and *PO*. If we imagine the tie *KL* to be cut in two just above the joint, the main rafters would break at the joints 4 and 9, and the bottom portion immediately slide outwards, straightening out the main tie and allowing the top of the truss to fall through.

Having seen that a hammer-beam truss *could* be built in which there is no horizontal thrust, we will now consider the hammer-

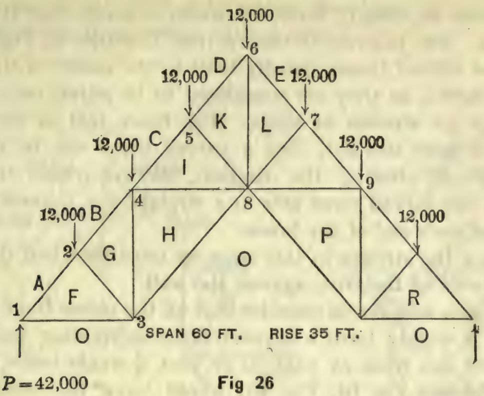


Fig 26

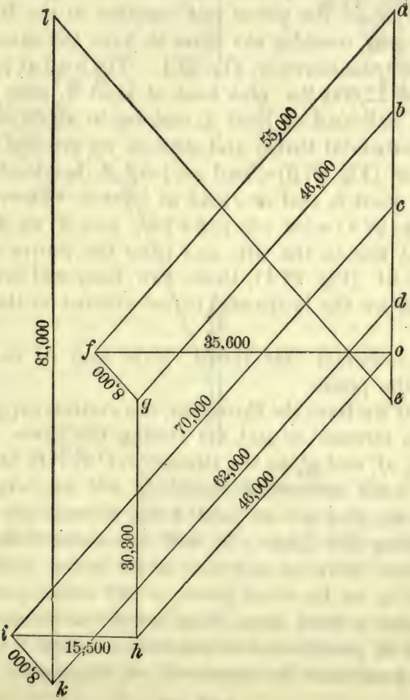


Fig. 26 A

beam truss as usually built, in which a horizontal thrust is expected. The diagram of such a truss is shown in Fig. 27, in which the curved braces usually built in the centre of the truss are not shown, as they are considered to be purely ornamental and have no stresses in them. The brace *OM* is drawn as though it were straight; but a curved brace can be used as well, without altering the diagram, for the reason that the stress in the curved piece acts in a straight line connecting the centres of each end of the brace.

To draw the stresses in this truss we must first find the horizontal thrust of the truss against the wall.

To do this we have to consider that all the pieces from joints 0 to joint 4 simply form a framed brace supporting the upper portion of the truss at joint 4, or that a single brace, shown by the broken line 04, Fig. 27, would have the same effect on the wall as all the pieces put together in the framed strut; that is, we may consider the truss to have the same horizontal thrust as the truss shown in Fig. 27*A*. The load at joint 4 would evidently be 12,000 lbs. plus load at joint 5, plus half-load at joint 6 and half-load at joint 2, making in all 36,000 lbs. To draw the horizontal thrust and stresses we proceed as follows:

Lay off *ab* (Fig. 27*B*) = load at joint 2, *bc* = load at joint 4, *cd* = load at joint 5, and *de* = load at joint 6. Then the load at joint 4 (Fig. 27*A*) $= \frac{1}{2}ab + bc + cd + \frac{1}{2}de$; and if we draw from *x* a horizontal line to the left, and from the centre of *ab* a line parallel to 04 (Fig. 27*A*), these two lines will intersect at *m*, and *mx* will be the horizontal thrust exerted on the wall at the point 0.

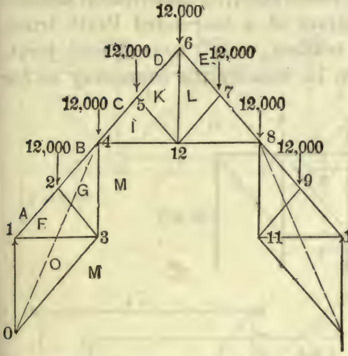
Having obtained this thrust, it is easy to determine the stresses in the pieces.

At joint 0 we have the thrust *mx*, the vertical supporting force *xa*, and the stresses *ao* and *mo* closing the figure. At joint 1 we have *oa*, *af*, and *of*, as the stresses in *OA*, *AF*, and *FO*.

At joint 3 the stresses are *mo*, *of*, *fg*, and *mg*; at joint 2 they are *fa*, *ab*, *bg*, and *gf*; at joint 4 the stresses are *mg*, *gb*; *bc* and *ci* closing the figure. It will be noticed that the figure closes without allowing any line to be drawn parallel to *MI*; hence there is no tensional stress in *MI* under purely vertical loads. Under a wind stress there would be some compression in *MI*, and in practice a tie-beam is necessary.

At joint 5 we have the stresses *ic*, *cd*, *dk*, and *ki*, and at joint

6 we have *kd*, *de*, *el*, and *kl*, which completes the stresses for one-half of the truss, which is all we need.



$P = 42{,}000$

Fig. 27

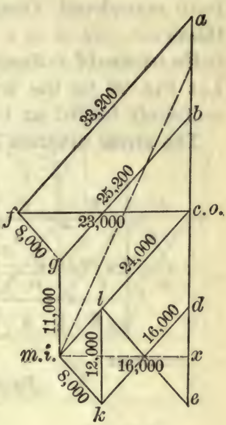


Fig. 27 B

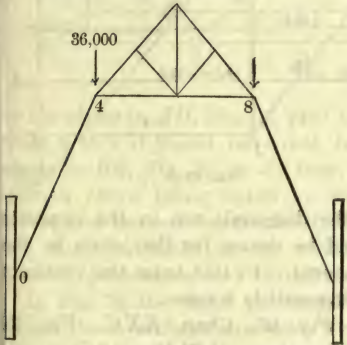


Fig. 27 A

Comparing, now, the diagram of stresses, Fig. 27*B*, with Fig. 26*A*, we find that in general the stresses in the truss, Fig. 27, are much less than in the truss, Fig. 26; while, on the other hand, the latter truss exerts no outward thrust on the walls, as is the case in Fig. 27.

By building a truss like Fig. 27, and putting in curved ties from joints 3 and 11 to joint 12, we can relieve the brace *OM* of part of the load without straining the other timbers as much as is the case in Fig. 26.

The truss shown in Fig. 36, Chap. XXV, combines the advantages of both the forms of hammer-beam trusses which we have considered, though it may not be quite so pleasing to the eye.

EXAMPLE 17.—*Suspended Pratt Truss, Symmetrically Loaded.* Let Fig. 28 be the truss diagram of a suspended Pratt truss, uniformly loaded at top and bottom, with 2 tons at each joint.

The stress diagram is drawn in exactly the same way as for

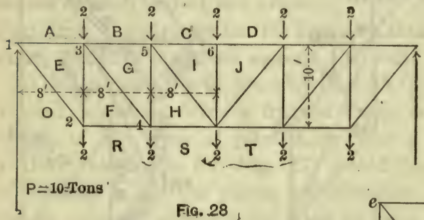


FIG. 28

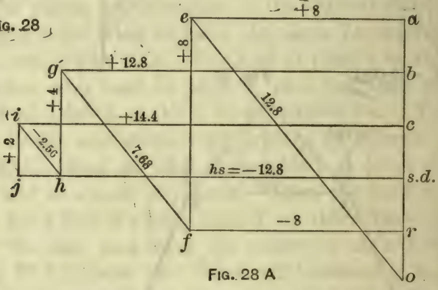


FIG. 28 A

a Howe truss, except that the diagonals run in the opposite direction. The stresses should be drawn for the joints in the order in which they are numbered. In this truss the verticals are in compression and the diagonals in tension.

EXAMPLE 18.—*Truss like Fig. 63, Chap. XXV.* Fig. 29 is the truss diagram of a light iron truss of 32 ft. span intended to support a tar-and-gravel roof, the slope of the roof being at right angles to the line of the trusses.

The stress diagram is drawn as in the previous examples, taking the joints in the order in which they are numbered.

EXAMPLE 19.—*Double Warren, or Lattice Truss.* The truss diagram shown by Fig. 30 is best analyzed by considering it as built up of two Warren trusses, laid one over the other, the full lines indicating a truss such as is shown in Fig. 31, and the dotted lines a truss like Fig. 32. Three of the seven

loads would come on the first truss and four on the second. The stresses are found for each truss separately and then combined for the top and bottom chords.

Thus the stress in the top chord, from 1 to 3, Fig. 30, would be that in AD, Fig. 31, or 3 tons; from 3 to 5 it would be equal

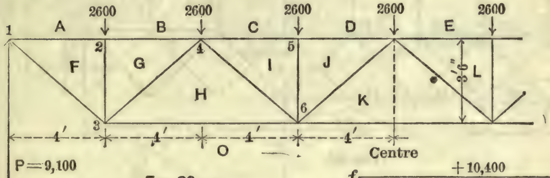


Fig. 29

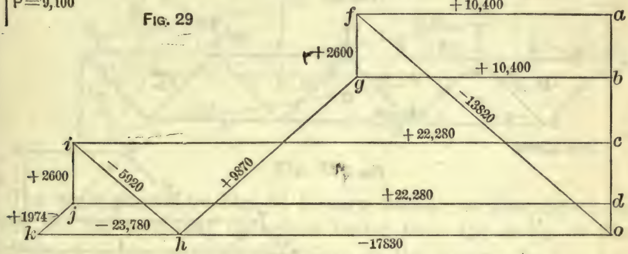


Fig. 29 A

to the stress in AD, Fig. 31, plus that in BE, Fig. 32, or 9 tons; from 5 to 7 it would be equal to stress in BF, Fig. 31, plus stress in BE, Fig. 32, or 13 tons, and so on, the stress in the bottom chord being found in the same way. The diagonal struts and ties act independently of each other, and the stresses are those indicated on the stress diagrams. The plus signs denote compression and the minus signs tension.

In Fig. 32 the stress polygon for joint 7 is fe, eb, bc, and cf, which closes without any room for a line parallel to FF, showing that there is no stress in the two inner diagonals except that due to the weight of the bottom chord.

This truss can be extended indefinitely by giving it sufficient height for the span. It is usually built of steel angles with riveted joints.

Example 20.—*Truss like Fig. 66, Chapter XXV.* Fig. 33 is the truss diagram of a truss similar to that shown by Fig. 66, p. 926, the panel loads being taken at 2 tons each, the analysis being the same for any other loads. The stress diagram is drawn exactly as in the previous examples, commencing with

the supporting force $=oa$ and taking the joints in the order in which they are numbered. In this truss the diagonal web members are all in tension and the verticals in compression. It will be noticed that the diagonals in the two panels nearest

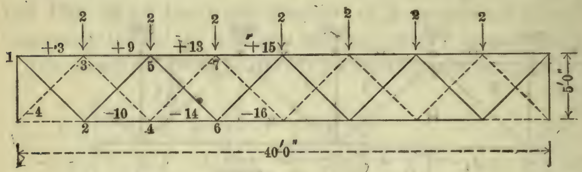


FIG. 30

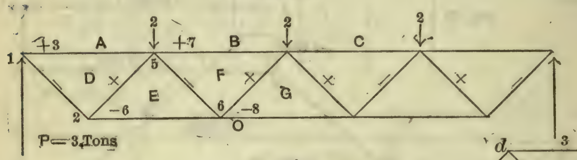


FIG. 31

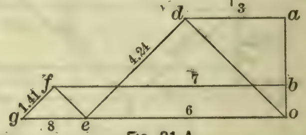


FIG. 31 A.

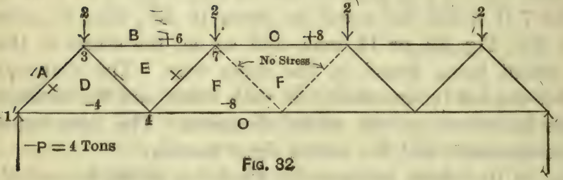


FIG. 32

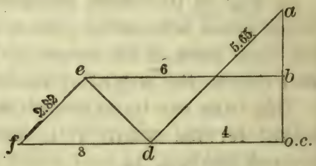


FIG. 32 A

the centre incline in the opposite direction from those in the outer panels. This is due to the inclination of the top chord, which would cause the inner diagonals to be in compression if they inclined the other way. The stress in LM, however,

is so small that a single angle would resist either a compressive or tensile stress.

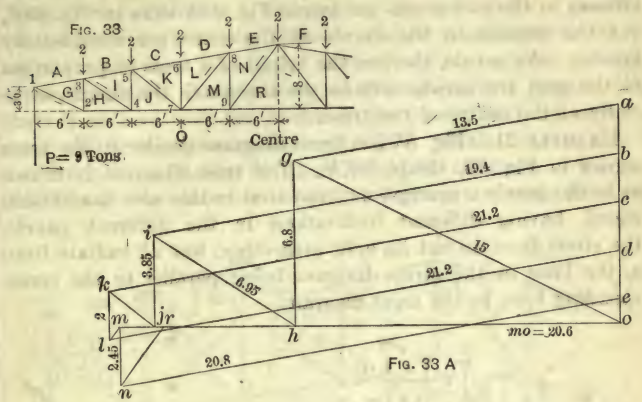


Fig. 33

The truss shown by Fig. 34 is very similar to that shown in Fig. 33, the principal difference being that the slope of the top chord is less in the former than in the latter. In Fig. 34, only the diagonals in the two centre panels incline from the

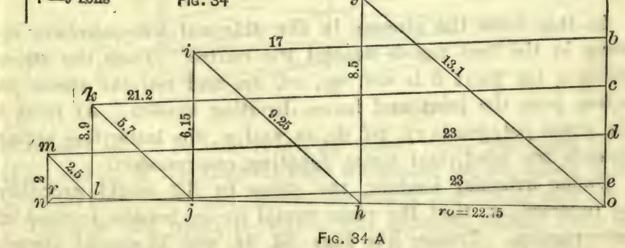


FIG. 34 A

centre, and the stress in these diagonals is very small. With a still less inclination to the top chord, the stress in NR would become 0, and with a horizontal top chord the stress in NR would become reversed, or to keep it in tension, the direction

should be changed, as in the Pratt truss, Fig. 28. Comparing the stresses in these two trusses, it will be seen that while the stresses in the end panels are less in Fig. 34*A* than in Fig. 33*A*, yet the stresses in the chords at the centre are considerably greater. As a rule, the less the height of a truss in proportion to the span the greater will be the stresses in the chords, especially at the centre of the truss.

EXAMPLE 21.—Fig. 35 is a truss diagram similar to the truss shown in Fig. 73, Chap. XXV. The truss diagram is drawn as in the previous examples, except that in this case the bottom chord, having different inclinations in the different panels, the stress lines do not lie over each other, but all radiate from *o*, the lines in the stress diagram being parallel to the corresponding lines in the truss diagram.

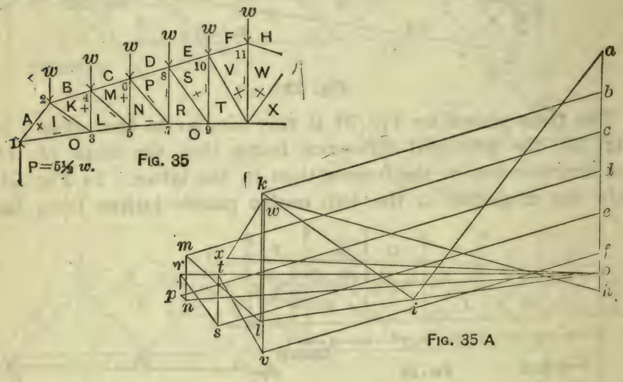


FIG. 35

FIG. 35 A

In this truss the stresses in the diagonal web-members reverse in the two panels nearest the centre. Thus the stress polygon for joint 6 is *nm*, *mc*, *cd*, *dp*, and *pn*, the stress *pn* acting *from* the joint and hence denoting tension. At joint 8 the stress polygon is *rp*, *pd*, *de*, *es*, and *sr*, the latter line acting *towards* the joint, and hence denoting compression.

Under irregular loading, the stress in *SR* would probably be reversed, so that the piece would be in tension instead of compression. Trusses like Figs. 33, 34, and 35 should always be computed for snow on one half of the truss only, and also for wind pressure.

EXAMPLE 22.—In Fig. 36 we have the diagram of the truss shown by Fig. 72, Chapter XXV. This truss is similar to

that shown by Fig. 34, except for the secondary bracing in the panels and for the curved tie-beam. The stress diagram presents no difficulties. In drawing the lines from *o* parallel to the bottom chord the latter should be considered as made up of straight lines joining the joints; thus *om* is drawn parallel to an imaginary straight line connecting joints 4 and 8. As there is no load over the centre of the panels next the centre, there will be no stress in XX' and $X'X''$. If the bottom chord was straight, as in Fig. 34, there would be no stress in the tie YZ, but as the chord is curved, it requires a certain amount

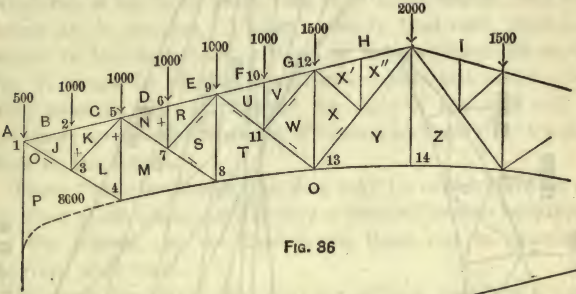


Fig. 36

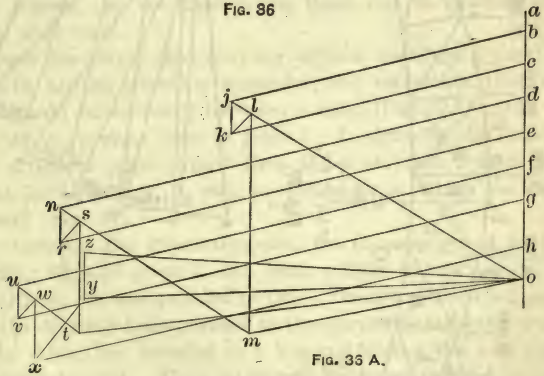


Fig. 36 A.

of tension in YZ to hold it up, the stress being indicated by *yz* (Fig. 36*A*). If the diagram were completed for the entire truss, it would be symmetrical about a horizontal line drawn through *o*.

There is one point in which this example differs from all of the preceding, viz., in the load AB over the support. As a matter of fact, there would be such a load over the support for all of the trusses, but as the load does not act on the truss, but only on the support, it has been neglected in the previous examples.

When trusses are supported by columns this outer load should be taken into account in computing the load on the

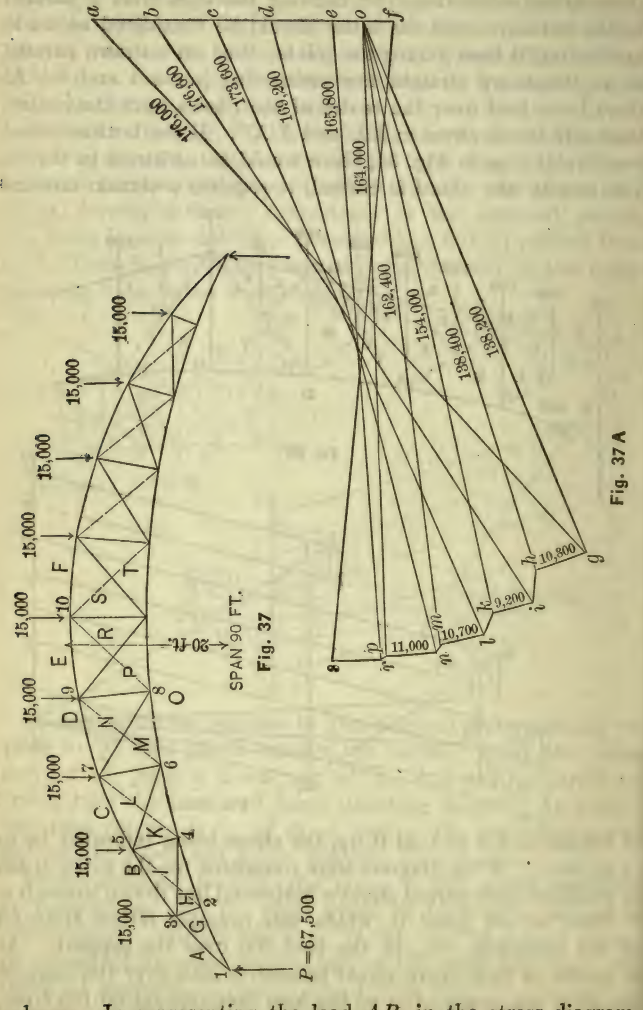


Fig. 37

Fig. 37 A

column. In representing the load AB, in the stress diagram, we first measure off on a vertical line, oa, equal to the support-

ing force P, which acts up, then measure down ab, equal to load AB; then draw bj and oj. The load AB has no effect on the stresses in the truss, because if it is omitted the value of P would be just that much less.

EXAMPLE 23.—*Iron Bowstring Truss.* Span of truss, 90 ft.; distance between trusses from centres, 20 ft.; rise of arched rafter, 20 feet.

The form of truss, represented by Fig. 37, is one of the most economical of trusses for very great spans.

In such cases as the present example, the rafter, or curved principal, is the only piece that is in compression, and all the others are in tension. Under a steady load only, such as the weight of the roof itself, one set of braces placed as shown in Fig. 37 would be all that would be needed; but under a severe wind pressure blowing against one side of the roof only, it is necessary to have another set of braces, as shown by the dotted lines in the figure.

These "counter-braces," as they may be called, have no stress on them at all when there is only a vertical load to be supported by the trusses; so we must leave them out in drawing the diagram of stresses.

To draw the stress diagram, lay off the loads on a vertical line, as in all the previous examples, and remember that the point o should be half-way between e and f (Fig. 37A); then oa will be the supporting force at joint 1. In drawing the stresses at the different joints, draw first those at joint 1 and then those at joints 2, 3, 4, 5, etc., in the order in which they are numbered (Fig. 37).

To commence the stress diagram, we have oa equal to the supporting force at joint 1, and from a draw a line parallel to AG, and from o a line parallel to OG. These two lines intersect at g. (In drawing lines parallel to the curved lines of the truss, draw the stress line parallel to a line connecting the two ends of the curved chord. Thus ag should be drawn parallel to 1–3, and og parallel to 1–2.) At joint 2 we already have og, and from g draw a line parallel to GH, and from o a line parallel to OH (2–4): this gives the lines oh and gh.

At joint 3 we have hg, ga, and the load ab, and draw bi and hi from b and h.

At joint 4 we now have oh and hi, and draw ik and ok. The stress lines for joints 6 and 8 are drawn in a similar way, and those for 5, 7, and 9 similarly to those at joint 3. After drawing

the stresses at joint 9, go to joint 10; and after drawing the lines for that joint all the stresses in the truss will have been obtained. The stresses in this particular example are given in pounds on the respective lines in the stress diagram. It will be noticed that the stress is very severe on the top and lower chords, but very slight on the bracing. It is in fact so slight that it will be about as well to make all the diagonal braces of the same size sufficient to resist the stress on IH, where the stress is the greatest; or IH and KL might be the same size and MN and PR a smaller size.

The vertical or radiating pieces might be all of a sectional area capable of resisting the stress in NP.

The great advantage of this truss lies in the fact that *all its parts are in tension* excepting the upper chord, which, of course, is in compression. We might analyze the way in which the stresses act, by saying that the upper chord carries all the load, like an arch, and is prevented from spreading out at the ends by the lower tie. The object of the bracing and vertical pieces is only to keep the tie in its curved position, and not permit it to come down flat, and thus allow the ends of the arch to spread out.

Trusses Unsymmetrically Loaded.—In all of the preceding examples the loads have been symmetrically disposed each side of the centre of the truss, so that each supporting force is equal to one half of the total load. It very often happens, however, that the loads are not symmetrically disposed, and in all such cases it is necessary to first compute the supporting forces and then to draw the stress diagram for the entire truss, as the diagram will not be symmetrical. Otherwise the process is the same as described for the foregoing examples. The following examples will illustrate sufficiently the method of computing the supporting forces and drawing the stress diagrams:

EXAMPLE 24.—Fig. 38 represents the diagram of a truss similar to that shown in Fig. 1, but of a greater span and having a gallery supported from it at one side only. The approximate roof and ceiling loads are indicated by the figures near the arrows, and the weight coming on one truss from the gallery would be about 9,000 lbs.

The first step towards drawing the stress diagram is to compute the reactions at the two ends of the truss, which will give the supporting forces. This is most readily done by the method

of moments explained on pages 274–277. In this example we will take the moments about joint 1.

As the loads at joints 2 and 3 have the same arm, we will add them together before multiplying by the arm, also the loads at joints 4 and 5, and 6 and 7. The moments about joint 1 will then be:

$$(8{,}000+4{,}500+9{,}000)=21{,}500\times12\tfrac{1}{2}=\ 268{,}750 \text{ ft.-lbs.}$$
$$(8{,}000+4{,}500)\qquad =12{,}500\times25\ =\ 312{,}500 \quad \text{"}$$
$$(8{,}000+4{,}500)\qquad =12{,}500\times37\tfrac{1}{2}=\ 468{,}750 \quad \text{"}$$

$$\text{Total moments}=1{,}050{,}000 \text{ ft.-lbs.}$$

This moment must be balanced by the supporting force P_2, which has an arm equal to the distance between P_1 and P_2, or 50 ft. Knowing the arm the force is obtained by dividing the total moment of the loads by the span. Dividing 1,050,000 by 50 we have 21,000 lbs. as the reaction or supporting force at joint 8, and P_1 must equal the difference between the sum of the loads and P_2. The sum of the loads is 46,500 lbs., and subtracting 21,000 we have 25,500 as the value of P_1. We are now ready to draw the stress diagram, Fig. 38*A*. First draw a vertical line $oa=P_1=25{,}500$ lbs. From *a* and *o* draw lines parallel respectively to *AE* and *OE*, which gives the point *e*. For the stress lines at joint 2 we measure up from *o* a distance equal to the load at that joint, 13,500 lbs., which gives the point *r*, and from *e* and *r* draw lines parallel to *EF* and *RF*, which intersect at *f*. At joint 3, the stress polygon is *fe*, *ea*, *ab*, *bg*, and *gf*. Draw the stress polygons for joints 4, 5, and 6 in the order in which they are numbered. At joint 6 the stress polygon is *ih*, *hc*, *cd*, *dj*, and *ji*. If the diagram has been correctly drawn, the line *ij* will be just equal to the load at joint 7. The stress polygon for joint 7 is $ts=4{,}500$ lbs., *si*, *ij*, and *jt*, the only line to be drawn being *jt*, which must be parallel to *JT*, consequently *j* must be exactly opposite *t*, or the polygon will not close. The distance *dt* should be equal to P_2.

In this example we have taken the ceiling and roof loads separately, and hence the full stress in the vertical members is shown by the stress diagram. It is simpler, however, to add the ceiling loads (including the weight from the gallery to the roof loads) and then draw the stress diagram as though the loads were applied at joints 3, 4, and 6.

The stress diagram, Fig. 38*B*, is drawn in that way. In this diagram the rods *EF* and *IJ* are not represented, be-

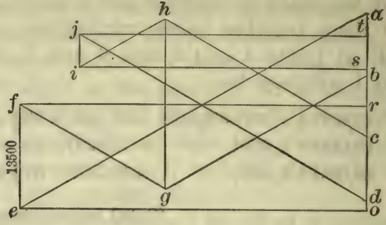


FIG. 38 A

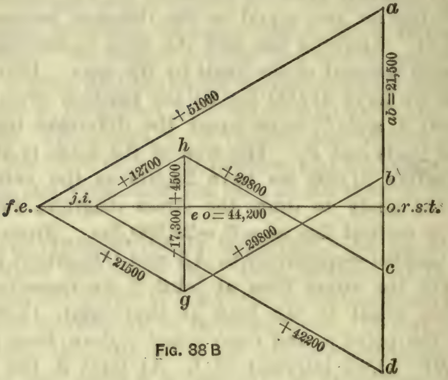


FIG. 38 B

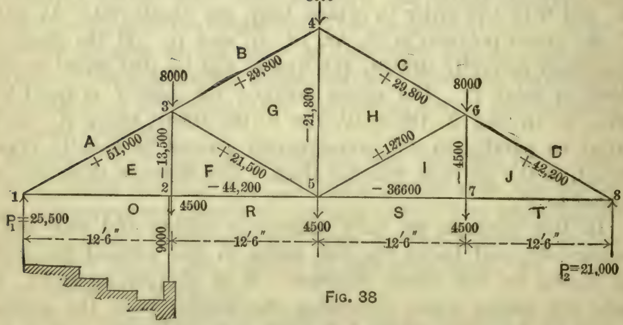


FIG. 38

cause they have no stress when the loads are all at the top. In drawing Fig. 38*B*, the last line drawn is *di*, which must

start from d and be parallel to JD. If it does not pass through the point i, previously found, then the diagram has not been correctly drawn, or else an error has been made in computing the supporting forces.

Comparing Figs. 38A and B, it will be seen that the inclined lines and also the lines representing the stresses in different parts of the tie-beam are of the same length in both diagrams, but the line gh in Fig. 38B is less in length than the corresponding line in Fig. 38A. This difference should be just equal to the load at joint 5, consequently if we draw the stress diagram as in Fig. 38B, we must add to the stress obtained by scaling the line gh, the load at joint 5. The stresses in the rods EF and IJ are just equal respectively to the loads at joints 2 and 7, as the only purpose of these rods is to transmit the loads at 2 and 7 to the joints above.

When these rods are inclined from a vertical, the ceiling loads must be treated separately as in Fig. 38A.

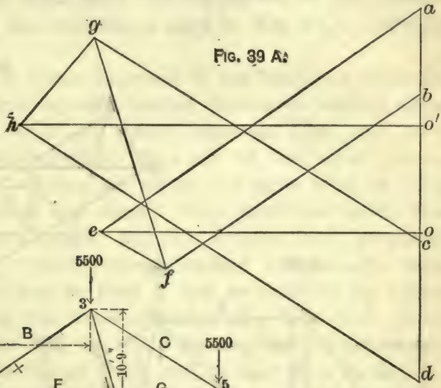


FIG. 39 A.

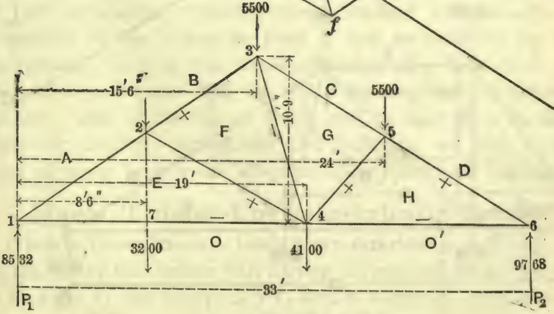


FIG. 39

EXAMPLE 25.—Fig. 39 is the diagram of a wooden roof truss designed by the author for a certain building. The actual

loads were about as given on the diagram. Purlins occurred at joints 3 and 5 only, and the ceiling below was suspended by rods from joints 4 and 7, joint 4 being fixed by the framing of the ceiling.

The moments of the loads about joint 1 are:

$$
\begin{aligned}
3{,}200 \times 8\tfrac{1}{2} &= 27{,}200 \text{ ft.-lbs.}\\
5{,}500 \times 15\tfrac{1}{2} &= 85{,}250 \quad \text{"}\\
4{,}100 \times 19 &= 77{,}900 \quad \text{"}\\
5{,}500 \times 24 &= 132{,}000 \quad \text{"}\\
\hline
\text{Sum of moments} &= 322{,}350 \text{ ft.-lbs.}
\end{aligned}
$$

Dividing the sum of the moments by the distance between supporting forces, we have 9,768 lbs. as the value of P_2. The sum of the loads is 18,300 lbs. Subtracting 9,768, we have 8,532 as the value of P_1.

To draw the stress diagram start with oa = 8,532 lbs. = P_1,

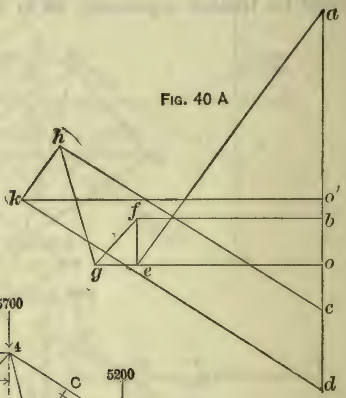


FIG. 40 A

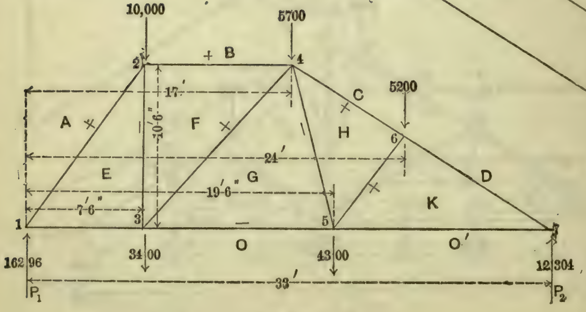


FIG. 40

and draw ae and oe. Assume that the load at 7 is transferred to joint 2, then the stress polygon for joint 2 is ea, ab, bf, and

fe. At joint 3 we have *fb*, measure down $bc = 5{,}500$ lbs., and draw *cg* and *fg*. At joint 4, start by measuring upwards from *o* 4,100 lbs., giving point *o'*, and draw *gh* and *o'h*. At joint 5 we have *hg*, *gc*, measure down $cd = 5{,}500$ lbs., and a line from *d* drawn parallel to *DH* should pass through *h*, which completes the diagram. The stress in rod 2–7 will be the load at joint 7.

EXAMPLE 26.—Fig. 40 is the truss diagram of another truss used in the same building as the truss shown by Fig. 39. Taking moments about joint 1, we have for the sum of the moments 406,050 ft.-lbs., and dividing by 33 ft., we have 12,304 as the value of P_2. The sum of the loads is 28,600 lbs., which leaves 16,296 lbs. for the value of P_1.

The stress diagram is drawn in the same manner as Fig. 39*A*, starting with $oa = P_1$. *ab* is drawn equal to the sum of the loads at joints 2 and 3, and the actual stress in *EF* is 3,400 lbs. plus the length of the line *ef*. If the stress diagram is correctly drawn a line through *d* parallel to *KD* will pass through the point *k*, previously obtained. The character of the stresses is indicated by the plus and minus signs on Fig. 40, + denoting compression.

If we compare the stress diagrams in the last three examples with those for symmetrically loaded trusses of similar shape we shall find that while the stress diagrams, Figs. 38*A*, 39*A*, and 40*A*, are unsymmetrical, they are of the same general character, and the stresses are all of the same kind as when the supporting forces are equal. This condition holds true for most triangular trusses, but for trusses with horizontal or curved chords, unsymmetrical loading will usually cause a reversal of the stress in kind in one or more of the braces or verticals, and if the truss contains any four-sided panels an additional brace will generally be required. This is particularly true of the Howe truss, and as this truss is very extensively used by architects and builders, we will now consider at some length the effect of unsymmetrical loading.

Howe Trusses Unsymmetrically Loaded.

When a Howe truss is loaded symmetrically each side of the centre, all of the braces will incline downward from the centre, as in Figs. 17–20, Chap. XXV, and if there are an odd number of panels, the centre panel will need no brace.

When a load of much magnitude is placed on one side of a truss having an odd number of panels without a corresponding

load on the other side, a brace will always be required in the centre panel and the brace should incline downward from the side which is most heavily loaded.

When the truss has an even number of panels, an unsymmetrical load will cause a greater stress in the braces on one side of the truss than on the other, and if there is a sufficient difference between the loads on one side of the truss from those on the other, it will cause a compression stress in one or more of the rods and a tensile stress in one or more of the braces. Now, as this truss is especially designed with the idea of having the braces in compression and the verticals in tension, whenever the loading would cause tension in a brace, or a compression in a rod, then the direction of the brace should be reversed, which will cause it to be in compression again.

For instance, take the truss, Fig. 41, having 6 equal panels

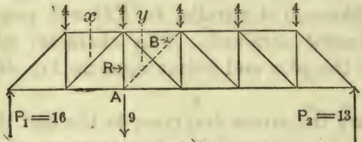


Fig. 41

and loaded with 4 tons at each of the upper joints and 9 tons at the second lower joint. Without the bottom load of 9 tons the brace in the third panel should incline downward from the centre joint, as shown by dotted line at *B*, but when the load of 9 tons is added it will cause a tensile stress in *B* and a compression stress in *R*. To avoid this, the direction of the brace is reversed as shown by the full line, and the brace is then in compression, and the vertical *R* has no stress except the direct load of 9 tons. The same thing would occur if the load of 9 tons was applied at the joint directly above, instead of at

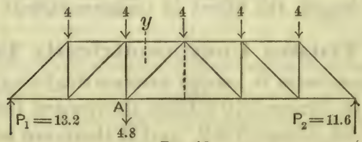


Fig. 42

the lower joint, although in that case there would be no stress at all on *R*, except the weight of the tie-beam. If the load of

9 tons were reduced to 6 tons, then no brace at all would be required in the third panel, and when the bottom load is less than 6 tons, then a brace in the normal direction is required, as shown in Fig. 42.

In the five-panel truss, shown by Fig. 43, a load of 7.5 tons at A would require the arrangement of braces shown by the full lines, and if the load at A were increased to more than 15 tons, then the brace R would need to be reversed, as shown by the dotted line.

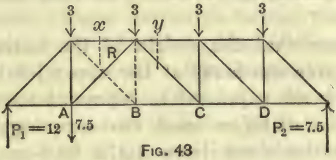


FIG. 43

The stress diagram will always show in which direction any brace should be placed to be in compression, but it may also be determined by the following:

Rule.—When the sum of the loads to the left of any section taken between P_1 and the centre is *greater* than the reaction at P_1, then the direction of the brace cut by that section must be reversed from its normal direction. When the sum of the loads is *less* than P_1, then the brace should be in its normal position. When the sum of the loads (to the left of the section) is just equal to P_1, then no brace at all will be required. For example, take a section at X, Fig. 43; here the sum of the loads to the left is 10.5, which is less than P_1, and consequently the brace should be in its normal direction.

If we take a section at Y, the sum of the loads to the left is 13.5 or greater than P_1; hence a brace will be required slanting downward from the heavier loaded side.

Taking a section at X, Fig. 41, the load to the left is 4, which is less than P_1; hence the brace in that panel should be in its normal position. Taking a section at Y, the sum of the loads is greater than P_1, and hence the brace in that panel must be reversed.

Taking a section at Y, Fig. 42, the sum of the loads to the left is less than P_1; hence the brace should be in its normal position. By the rule above given, one can always tell in which direction the brace in any panel should be placed, no

matter how complicated the loading, or whether or not the panels are of equal width; but to apply the rule, it is first necessary to determine the supporting forces, which can be done by the method of moments explained in Example 24.

EXAMPLE 27.—As an example of an unsymmetrical Howe truss unsymmetrically loaded, we will take the truss represented by the diagram, Fig. 44. This truss is supposed to support a ceiling over a hall, a flat roof, and a wooden tower located as shown. The position of the tower necessitates a division of the panels as indicated, so that the truss is quite unsymmetrical.

We will assume that the weight of the ceiling, roof, snow, and tower will give the loads at the upper joints indicated by the figures which are supposed to represent tons. Multiplying each load by its distance from the support P_2 and adding together the products we have 1,475 foot-tons as the total moment. Dividing by the distance between P_1 and P_2 (62), we obtain 23.8 as the value of P_1. The total load is 45.5 tons; hence P_2 will equal 21.7 tons.

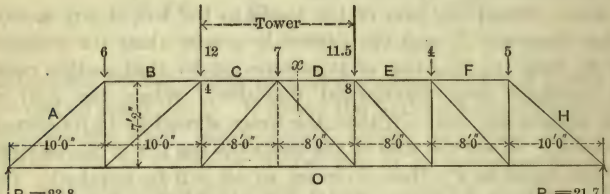


FIG. 44

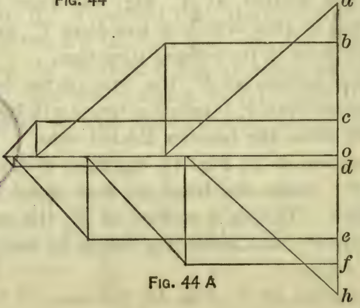


FIG. 44 A

The only panels of this truss in which there would be any question as to the direction of the braces are the third and

fourth. Taking a section at X, the sum of the loads to the left is greater than P_1; hence the brace should be placed as drawn. A section taken through C would give the sum of the loads to the left less than P_1, and hence the brace should be in its normal position. The stress diagram of this truss is readily drawn, starting with $oa=P_1$ and going from joint to joint as in previous examples. The completed stress diagram is shown by Fig. 44*A*. To the stresses on the verticals obtained from the diagram 44*A* should be added the ceiling loads which they support.

Counter Braces.—These are extra braces that are put in a truss to counteract the effect of a load which may be applied for a time and then removed. For illustration, let us consider the truss represented by Figs. 41 and 42. Here we have already seen that when the load at A is less than 6, the brace in the third panel should be in the position shown by Fig. 42, while when the load is greater than 6, the brace should be in the position shown by the full line, Fig. 41. Now, if the load at A represented the weight of a gallery with people in it, or a hoist for raising heavy loads, or in fact almost any live load, it is evident that when the live load was absent the brace in the third panel would need to be in its normal position, and when the full load is present a brace is needed in the opposite direction, and as it is not practical to move the brace to suit the condition of loading, it is necessary to put in two braces, only one of which, however, would come in action at any given time.

The stresses in a Howe truss, therefore, that are subject to a variable and unsymmetrical load should be computed for at least two conditions of loading, viz.: *a*, when the maximum load is applied; and, *b*, when the variable load is removed and the truss proportioned to resist both conditions. Snow is a variable load, to which such trusses are often subjected, but as it is nearly uniformly distributed over the roof, it would not change the stress (in kind) in any of the members; hence if the truss is designed for the maximum snow load, it will be more than strong enough when there is no snow. Moreover, the transverse strength of the chords is usually sufficient to resist any slight inequality in the loading.

The principal variable loads, therefore, to which a roof truss may be subjected that would require counter braces are those due to the weight of people, merchandise, etc., these either

being suspended from the truss by rods or brought upon the truss by a floor supported by the tie-beam. The truss shown in Fig. 44 is also an instance of such loading. The weights given by the figures indicate merely the combined dead and snow loads. During a high wind the weight on the leeward side of the tower would be much increased and lessened on the windward side, so that with the wind blowing from the right, the load at 4 would be much greater than indicated, and less at 8, while with the wind in the opposite direction the load would be increased at 8 and lessened at 4. This would require counter braces in both the third and fourth panels.

As counter braces can do no harm, even if they are never brought into action, it is always well to use them in the centre panels wherever the loads are at all variable.

Simple Cantilever Trusses.

Cantilever trusses may be considered as unsymmetrically loaded trusses, for although the loads may be symmetrical in relation to the truss, they are usually unsymmetrical in relation to the supports.

The method of computing the supporting forces and drawing the stress diagram is quite clearly shown by the following examples:

EXAMPLE 28.—Fig. 45 is the diagram of a cantilever truss such as might be used to support the roof over a grand-stand or depot-platform, and could be constructed either of wood or steel, although steel would be preferable.

The first step towards determining the stresses is to find the supporting forces.

For this purpose we have taken the panel loads at 1,000 lbs., all of the panels being of equal width, as this illustrates the method as well as actual loads and simplifies the problem.

In cantilever trusses the loads at the ends of the trusses should be taken into account as well as the intermediate loads. These end loads are each equal to one half of the panel loads.

To find the supporting forces, take moments about joint 13. The sum of the moments will be found to be 147,000 ft.-lbs. This moment must be resisted by the force P_1, which acts with a lever arm of 24′. Dividing 147,000 by 24, we have 6,125 as the value of P_1, and as the total load is 7,000 lbs., P_2 must be 875 lbs.

The stress diagram may be commenced either with the forces at joint 1 or those at joint 13, but as we have commenced at the left in all preceding examples we will continue in this order.

Commencing then with joint 1, we lay off on a vertical line the load $oa = 500$ lbs., which acts down, and from o and a draw lines parallel respectively to OI and AI. The forces act from o to a, a to i (from the joint), and from i to o (towards the joint), showing that AI is in tension and OI in compression, the reverse of a truss supported at both ends.

Next, at joint 2, we have the stress ia, and measure down $ab = 1{,}000$ lbs.; then draw ij and bj, IJ being in compression.

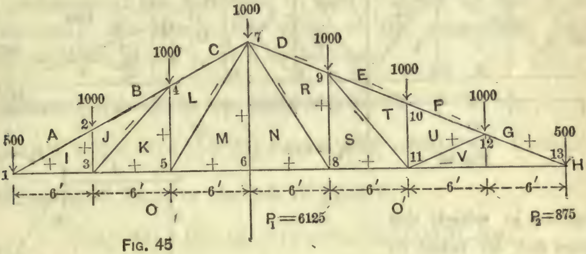


FIG. 45

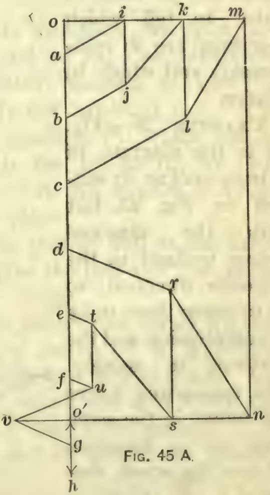


FIG. 45 A.

Next draw the forces for joint 3 and for the remaining joints in the order in which they are numbered. At joint 6, the first force which we know is the supporting force P_1, which we represent by measuring down from o, 6,125 lbs. (although the force acts up), giving the point o'. The polygon of forces will then be $o'o$, om, mn, and no'. It will be noticed that the stress in MN is equal to the supporting force, which is evident from the truss diagram. In practice, P_1 would probably be a post, which would be continued to the apex of the truss.

At joint 12 we have the stresses vu, uf, $fg = 1{,}000$ lbs., and

gv must close the polygon. It will be noticed that *gv* acts toward the joint; hence the rafter in the end panel is in compression. If a line drawn from *g* parallel to the rafter passes through *v*, then the stress diagram must be correct; if it does not pass through *v*, then either the stress diagram has not been drawn with sufficient accuracy or an error has been made in computing the supporting forces. In drawing the stress diagram for cantilever trusses, it is important to keep the

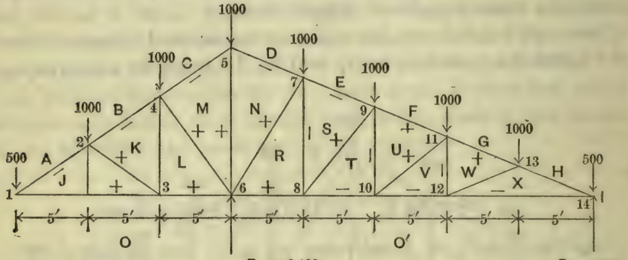


FIG. 46

direction in which the forces act in mind in order to tell which members are in compression and which in tension.

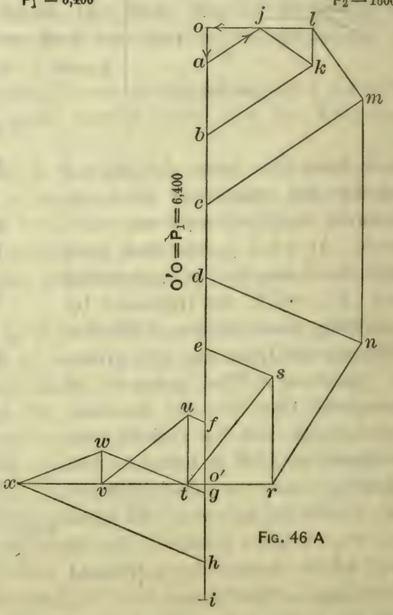


FIG. 46 A

EXAMPLE 29.—Fig. 46 is the diagram of a truss similar in outline to Fig. 45, but with the diagonal braces inclined in the opposite direction, so as to cause them to be in compression and the verticals in tension. The supporting forces are found in the same way as in Example 28, and the stress diagram is also drawn in the same way as Fig. 45*A*. In this truss, however, the stress

in the vertical post MN is considerably less than the reaction P_1, because a large portion of the loads is transmitted to joint 6 by the struts LM and NR. It will also be noticed that in this truss three sections of the rafter on the right side are in compression and three sections of the tie-beam in tension, owing to the fact that in this truss the projection of the overhang in proportion to the anchor span is less than in Fig. 45. It may be noticed that when the stress lines pass to the left of the load line (Fig. 46A) the stresses are reversed in kind.

This truss is better adapted to wooden construction (with vertical rods) than is the truss shown by Fig. 45.

EXAMPLE 30 (FIG. 47).—In this example we have a truss with an anchorage at the outer end to hold it down, so that P_1 acts downward. To obtain the value of the supporting forces take moments about joint 6 as follows:

Moments of loads to the right of joint 7:

$$(5\times 8)+(5\times 16)+(5\times 24)+(12.5\times 32)=640{,}000\text{ * ft.-lbs.}$$

Moments of loads to the left of joint 7:

$$(2.5\times 24)+(5\times 16)+(5\times 8)=180{,}000\text{ ft.-lbs.}$$

As these moments act in opposite directions, we must subtract the smaller from the larger, and we have an unbalanced moment of $640{,}000-180{,}000=460{,}000$, tending to turn the truss down on the right or to lift the left-hand end. This moment must be resisted by the reaction P_1, which has an arm of 24 ft. Dividing 460,000 ft.-lbs. by 24 ft., we have 19,250 lbs. as the reaction at P_1, or it will require a weight of this amount to hold the truss in place.

As the support P_2 must resist this pull as well as the loads, P_2 will equal the sum of the loads plus the pull at P_1, or $45{,}000+19{,}250=64{,}250$ lbs.

Having obtained the value of the supporting forces we proceed to draw the stress diagram by laying off on a vertical line $oa=19{,}250$ lbs. $=P_1$, remembering that it acts *down*. The next force is the load of 2,500 lbs., which also acts down, and which gives the point b; then from b draw a line parallel to BI and from o a line parallel to OI, and we obtain the point i.

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* Note that loads are in *thousands* of pounds.

bi acts from the joint and *io* towards it, showing that *BI* is in tension and *IO* in compression. The remainder of the stress

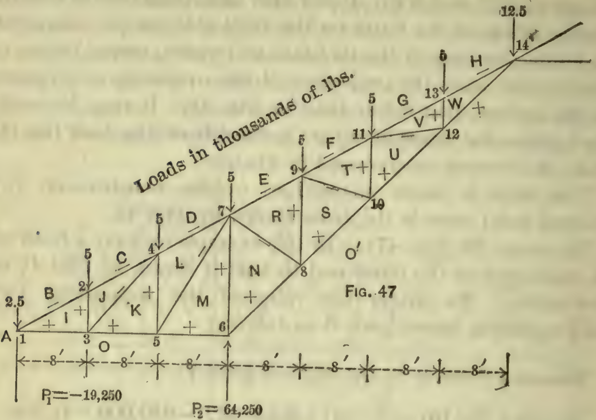


FIG. 47

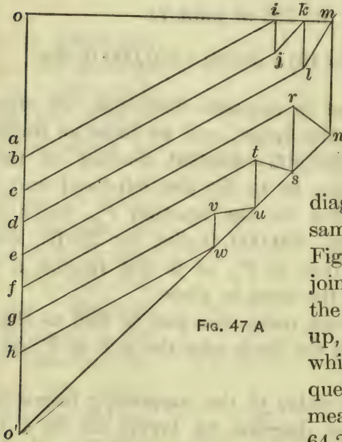


FIG. 47 A

diagram is drawn in the same way as diagrams Figs. 45*A* and 46*A*. At joint 6 we must start with the force P_2, which acts up, and the upper end of which must be at *o*; consequently we obtain *o′* by measuring down from *o*, 64,250 lbs.; the stress polygon for this joint being *o′o*, *om*, *mn*, and *no′*. After we have measured off *gh* = load at joint 13, the remaining distance *ho′* should be just equal to the load at joint 14, or 12,500 lbs. If P_1 and P_2 have been correctly computed and the stress diagram accurately drawn the points *s*, *u*, and *w* will come in the line *o′n*.

In the last three examples we have considered vertical loads only. Cantilever roof trusses, however, should always be computed for wind loads as well as for vertical loads and should be braced from the supports.

Three-hinged Arched Trusses.

Several examples of this type of truss are illustrated in Chapter XXV.

For computing the stresses each half truss is considered as an entire truss itself. Considering the half arch, Fig. 48, it is evident that a horizontal pressure must be exerted at the top to prevent the arch from falling down, and this horizontal thrust takes the place of one supporting force, the other being exerted on the bottom pin. In the actual truss this horizontal thrust is provided by the opposite arch, each half arch holding up the other.

In accordance with the mechanical principle that for a body to be in equilibrium the algebraic sum of the forces acting in any given direction on the body must equal zero, to maintain the half arch, Fig. 48, in equilibrium, a horizontal resistance must be exerted on the bottom pin equal to the horizontal reaction on the top pin, and the two horizontal forces must act in opposite directions. In practice the horizontal resistance at the bottom is usually provided by rods or eye-bars connecting the pins of the two half arches, although the resistance may be provided by an abutment as with a masonry arch.

In Fig. 48 the horizontal reactions are represented by the arrows H, H.

The vertical stress diagram for this truss is very readily drawn after the reactions are determined. These reactions consist of a vertical resistance, P_1, equal to the entire load on the half arch, and a horizontal resistance H. *To compute the horizontal resistance, obtain the algebraic sum of the moments of the loads about the bottom joint and divide by the vertical distance between the pins.* In obtaining the sum of the moments, those which tend to turn the truss to the right should be marked plus, and those which tend to turn the truss to the left minus.

EXAMPLE 31.—The moments of the loads in Fig. 48 about the bottom pin are as follows, commencing with the load at joint 5:

500 lbs.	× 3′ 6″ =		−1,750
1,000 "	× 7′ 6″ =	+7,500	
1,000 "	×18′ 6″ =	18,500	
1,000 "	×29′ 6″ =	29,500	
1,000 "	×40′ 6″ =	40,500	
1,000 "	×51′ 6″ =	51,500	
1,000 "	×62′ 6″ =	62,500	
1,000 "	×73′ 6″ =	73,500	

Total moment = 283,500 − 1,750 = 281,750

$H = 281,750 \div 72.5 = 3,886$, or say 3,890 lbs.

P_1 = sum of the loads = 7,500 lbs.

Having obtained P_1 and H, commence the stress diagram by drawing the vertical line $oa = P_1$ and a horizontal line from $o = 3,890$. The stress lines for the bottom joint will then be *so, oa, aj*, and *js*. Both *aj* and *js* act towards the joint, hence both of these members are in compression. At joint 1 the stress lines are *ja*, *ak*, and *kj*, the point *k* being obtained by drawing a line from *j* parallel to *JK*. At joint 2 the stress lines are *sj*, *jk*, *kl*, and *ls*. *jk* and *kl* act from the joint and *ls* towards the joint, hence *JK* and *KL* are in tension and *LS* in compression.

At joint 3 the stress lines are *lk*, *ka*, *am*, and *ml*, *am* acting from the joint, showing that *AM* is in tension. At joint 4 the stress lines are *sl*, *lm*, *mn*, and *ns*. At joint 5 we have *nm*, *ma*, measure down *ab* = 500 lbs., and draw *bp* and *np*, parallel respectively to lines *BP* and *NP*. Continue in the same way at all of the joints in the order in which they are numbered. In this example the point *c* happens to come very close to the point *k*, but it is merely a coincidence. The line *xy* is also very short, barely long enough to indicate the direction in which the stress acts.

At joint 20 the stress lines are *sd′*, *d′e′*, and *e′s*. Now if the horizontal resistance *H* was correctly computed and the stress diagram has been drawn with great accuracy, a line through *o* parallel to *IE′* will just pass through *e′*. Owing to the fact that the lines of the truss are at so many different angles, it is more than likely that when the stress diagram is completed the line *oe′* will not quite pass through *e′*, and it may be necessary to go over the diagram a second time with great accuracy to make it come out right.

In drawing the stress diagram for a truss of this kind it will

be necessary to keep in mind *the direction* in which the stresses act at each joint, in order to tell which members are in compression and which in tension, as a slight change in the pro-

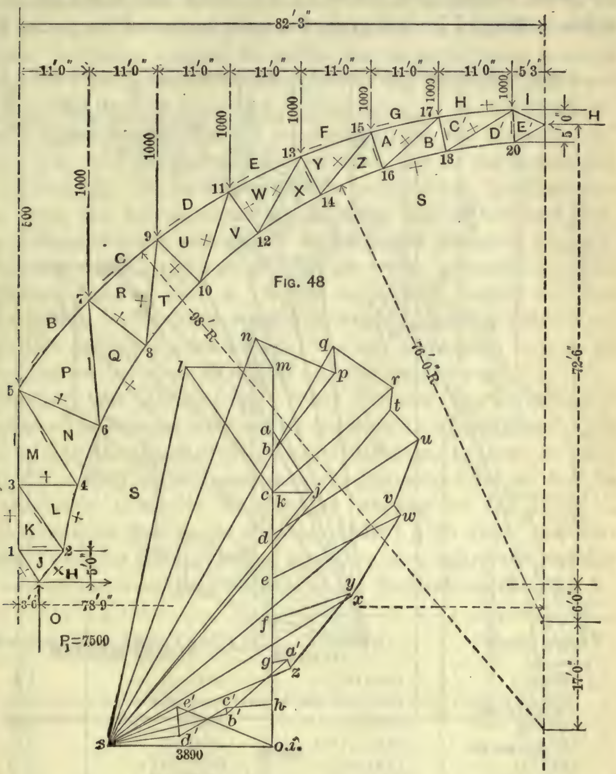


FIG. 48

FIG. 48 A.

portions of the arch or the location of the joints may change the character of the stress in the braces.

EXAMPLE 32.—Fig. 49 represents the truss diagram of one of the three-hinged arches used in the Liberal Arts Building of the Columbian Exposition at Chicago, 1893, the diagram being taken from one published in the *Engineering Record* of July 9, 1892. The trusses were spaced 50 feet apart, with trussed purlins supported at every other joint, as shown by

the arrows, Fig. 49. The *horizontal* roof area supported at each of joints 11, 15, 19, etc., was therefore 50′×38′, or 1900 square feet. The dead load was assumed at 42 lbs. per *horizontal* square foot (snow 12 lbs., steel and wood 30 lbs.), which multiplied by 950 gives the loads indicated for joints 11,

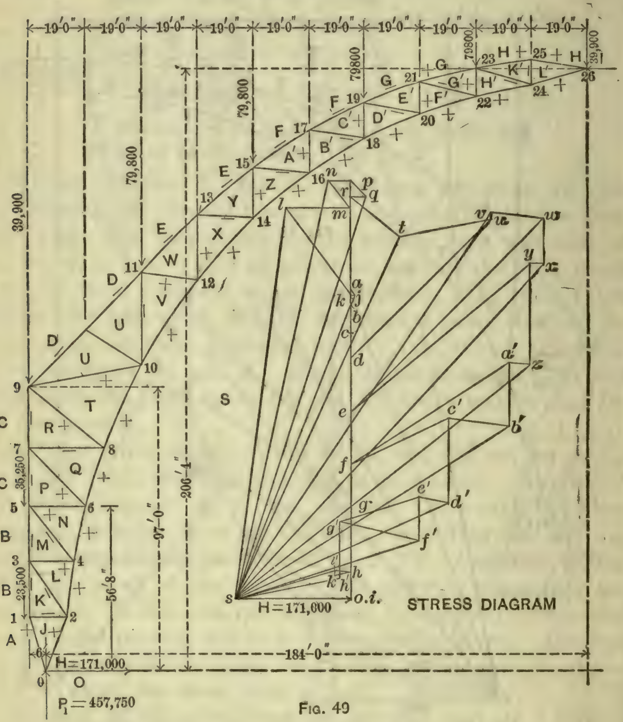


FIG. 49

15, etc. The loads at joints 9 and 26 would obviously be one half of the loads at intermediate joints. There are also dead loads at joints 1 and 5, due to the weight of outside walls and galleries. The total moments of the loads about the bottom joint, remembering that the moments for the loads at joints 1, 5, and 7 are negative, is 35,158,500 lbs.

Dividing this moment by the vertical distance between pins (206′ 4″), we have 170,400 lbs., or say 171,000.

The sum of the loads is 457,750 lbs. $=P_1$. The stress dia-

gram is drawn in the same way as explained for Fig. 48*A*, and presents no difficulties except in drawing the lines exactly parallel to the corresponding members of the truss. It will be noticed that the line *UU* has no stress because there is no load or other force to produce a stress where it joins the rafter. It was doubtless inserted to stiffen the rafter between joints 9 and 11. The character of the stress in the different members is indicated by the plus and minus signs, + in all cases denoting compression. It will be noticed that the stress in *AJ* and *JK* is very small under a dead load, due to the inclination of *AJ* and *JS*. In the stress diagram published in the *Engineering News*, the line *aj* comes on the other side of the load line, the difference being due either to the truss diagram, Fig. 49, not being exactly right or possibly to what appears to be an additional load at joint *o*. It will be noticed that if the line *JS* was but a very little steeper it would bring the point *j* on the other side of the load line. The stress diagram in Fig. 49 is correct for the truss as drawn and for the loads given.

Necessity for Determining Wind Stresses in Three-hinged Arches.—The stresses produced by the wind in trusses of this kind will in many instances greatly exceed those due to the dead load, and will also in many cases be of the opposite kind, so that it is absolutely necessary to compute stresses for the wind in both directions (see pages 1024 and 1029). To show how the stresses vary for dead and wind loads, we give below the stresses for several members, as published in the *Engineering Record:*

Member.	Dead Load.	Wind Left.	Wind Right.
AJ	− 11,000	+290,000	− 78,000
JS	+498,000	−228,000	+183,000
JK	+ 2,500	− 86,000	+ 24,000
BK	− 34,000	+277,000	− 75,000
KL	−168,000	+208,000	− 97,000
LS	+612,000	−389,000	+254,000
LM	+ 95,000	−119,000	+ 55,000

With wind to the right, piece *AJ* would have a total tensile stress of 89,000 lbs., and with wind from the left a compressive stress = 290,000 − 11,000, or 279,000 lbs. Piece *LS* would be subject to a compressive stress of 866,000 lbs. when wind is from the right and 223,000 lbs. when wind is from the left.

Wind Load Stresses.

Thus far we have considered the stresses due to vertical loads only, the pressure of the wind being combined with the dead load and considered as acting vertically. For triangular and Fink trusses this method is sufficiently accurate, as the wind pressure never causes a maximum stress in excess of that obtained by the method explained in connection with the foregoing examples. For trusses with curved chords, and in fact for almost all forms of steel trusses except the Fink and fan types, it is not safe to consider wind pressure as acting vertically, because the wind acts in a direction at right angles to the roof surface, and upon but one side of the roof at a given time, thus loading the truss unsymmetrically and often causing stresses of an opposite kind from those produced by a vertical load. Braces which are inactive under a vertical load may therefore be necessary to resist the force of the wind or the total stress due to wind and vertical load combined may be greater than it would be if the wind pressure were considered as a vertical load. To design a roof truss correctly, therefore, it is necessary to determine the stresses due to vertical loads and wind loads separately and then combine them so as to get the greatest stress that may be produced under any probable conditions.

In the calculation of trusses with curved chords it is the usual practice to find the stresses for the following different loadings and then combine them to obtain the maximum stress:

Stresses due to wind on the side of the truss nearer the expansion end, and for the wind on the side of the truss nearer the fixed end.

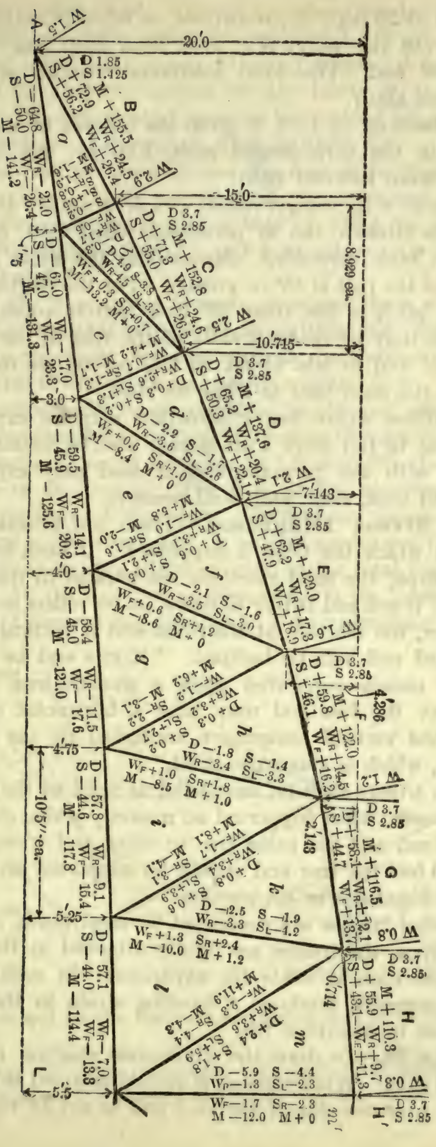
Stresses due to permanent dead load.

Stresses due to snow covering the entire roof or only one half, and even in special cases only a small area on one side.

It is generally assumed that the maximum wind pressure and the snow load can *not* act on the same half of the truss at the same time. Fig. 50 (from "Modern Framed Structures"), which is a half diagram of the roof trusses of the St. Louis Exposition Building,* shows the different stresses as figured for those trusses. All loads and stresses are given in thousands of pounds. The letters in connection with the stresses have the following significance:

D, permanent dead load at 20 lbs. per square foot; *S*, uniform

* Erected several years ago.



A
B
C
D
E
F
G
H
H'
L
W 1.5
W 2.9
W 2.5
W 2.1
W 1.6
W 1.2
W 0.8
W 0.8
20.0
15.0
10.715
7.143
8.929 ea.
4.286
2.143
0.714
3.0
4.0
4.75
5.25
5.5
10'5'' ea.

Fig. 50

snow load at 20 lbs.; S_L, snow load on left side only; S_R, snow load on right ride only; W_F, wind from fixed end; W_R, wind from roller end. The total maximum stresses are marked M (for each kind).

In the table on p. 1023 is given the stresses in a few of the members in the three-hinged arch, Fig. 49, due to vertical load, and wind left and right.

For trusses with *straight rafters* it will generally be sufficient to find the stresses due to permanent dead load, and to the wind from both directions, disregarding the snow load when the pitch of the roof is 45° or greater. For the Northern States when the pitch is less than 30° it is well to consider that a heavy sleet may be on both sides of the roof at the time of a heavy wind and to add about 10 lbs. per square foot of roof surface to the dead load to allow for sleet.

In localities where heavy snowfalls may be expected the stresses due to full snow load should also be found, as these combined with the permanent dead load may exceed those due to dead load, sleet, and wind pressure.

Wind Stress Diagrams.—These are affected by the manner in which the truss is supported. If both ends of the truss are fixed, the wind reactions are parallel to the resultant wind load; if one end is free to move, i.e. on rollers or supported on a rocker, the reaction at the roller end is vertical and that at the fixed end will be inclined. "If one end be fixed and the other merely supported upon a smooth iron plate, the reaction at the free end may have a horizontal component equal to the vertical component multiplied by the coefficient of friction, which is about one-third."

Wooden trusses may be considered as fixed at the ends.

Steel trusses, when supported on masonry walls, should have one end fixed and the other free to move and when the span exceeds 70 feet the free end should be supported on rollers to permit of expansion or contraction.

When steel trusses are supported by steel posts, as in steel mill buildings, the trusses are rigidly attached to the columns and no provision is made for expansion. In such buildings the wind pressure produces a bending strain in the columns which must be provided for.

EXAMPLE 33.—To draw the wind-stress diagram for a truss with fixed ends. Wind pressure is usually assumed to be applied uniformly over one side of the roof and to act at right angles

to the surface of the roof. The joint, or panel, loads will therefore be proportional to the roof areas supported. When the joints divide the rafter into panels of equal length, then the joint loads will be uniform, except for the joints at the edges of the roof. The actual wind pressure is obtained by multiplying the roof surface by the values given in Table IX.

For this example we will take the triangular truss shown in outline by Fig. 51 and assume that the span and spacing of the truss are such as will give a load of 1,000 lbs. at joints 2 and 4. The loads at joints 1 and 5 would be only one half of those at 2 and 4.

To find the supporting forces or reactions, draw a line representing the resultant of the loads, cutting the bottom chord at X. As the loads are symmetrical the resultant must act at the centre of the rafter and at right angles to it.

The reactions will be proportional to the two segments into which a horizontal line joining the points of support is divided by the resultant, or in this case to $1X$ and $X7$, the larger reaction being at joint 1. The sum of the reactions must be equal to the sum of the loads. To find the reactions graphically, draw a line from joint 1, at any angle, say from 30° to 45°, and measure off a distance equal to the total load.

In Fig. 51 the line 1–8 represents 3,000 lbs. Join 7 and 8, and from X draw a line parallel to 7–8, intersecting 1–8 at X'.

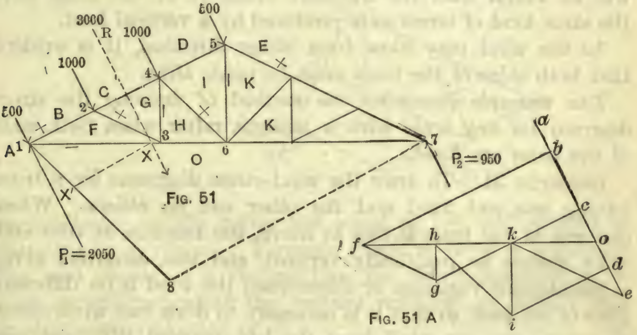


FIG. 51

FIG. 51 A

Then X'–8 will be the reaction at joint 1 and X'–1 the reaction at joint 7.

To draw the stress diagram, Fig. 51*A*, first draw the load line *ae* equal to the sum of the loads (in this case 3,000 lbs.)

and perpendicular to the rafter 1–5, and divide it so that $ao = X'$–8. Then at joint 1 we have oa equals the supporting force, $ab = 500$ lbs., and from b and o draw lines parallel respectively to BF and OF, intersecting at f. The stresses act in the direction oa, ab, bf, and fo, showing that BF is in compression and FO in tension.

At joint 2 the stress lines are fb, $bc = 1,000$ lbs., cg, and gf. The stress lines at joint 3 are of, fg, gh, and ho. At joint 4, hg, gc, cd, di, and ih. At joint 5, id, de, ek, and ki. (Note. If the load line has been correctly divided at o, and the stress lines drawn exactly parallel to the lines of the truss, the point k will be exactly above the point i.)

At joint 6 the stress lines are oh, hi, ik, and as the figure must close by a horizontal line through o, it is evident that the line KK of the truss diagram cannot be represented, and therefore there can be no stress in this member when the wind is from the left. At joint 7 we have the reaction eo, acting up, ok and ke must close the figure, showing that the line ke represents the stress in the entire length of the rafter to the right, and that there is no stress in the bracing on that side of the truss when the wind is from the left. If, however, either the lower tie or the rafter were not straight, some of the braces on that side would come into action.

By following the direction of the stresses in Fig. 51A, it will be found that the different members of the truss have the same kind of stress as is produced by a vertical load.

As the wind may blow from either direction, it is evident that both sides of the truss must be made alike.

This example illustrates the method of drawing the stress diagram for any truss with a straight rafter when both ends of the truss are fixed.

Example 34.—To draw the wind-stress diagrams for a truss having one end fixed and the other end on rollers. When one end of the truss is free to move, the reaction at that end must always be practically vertical, and this condition gives a considerable variation of stress when the wind is on different sides of the roof, so that it is necessary to draw two wind-stress diagrams, one for wind from the left, marked WL, and one for wind from the right, marked WR. It is customary with authors when writing on this subject to consider that the rollers are always under the *right-hand* support, and we shall follow this custom. In practice the rollers may be placed under

either end, as both sides of the truss are usually proportioned to the maximum stresses.

For this example we will take the same truss diagram that was used in Fig. 51, illustrating it again in Fig. 52, which is drawn to show wind from the left. Lay off the load line 1–8 and divide it at X', as in Example 33, and draw a line at ae,

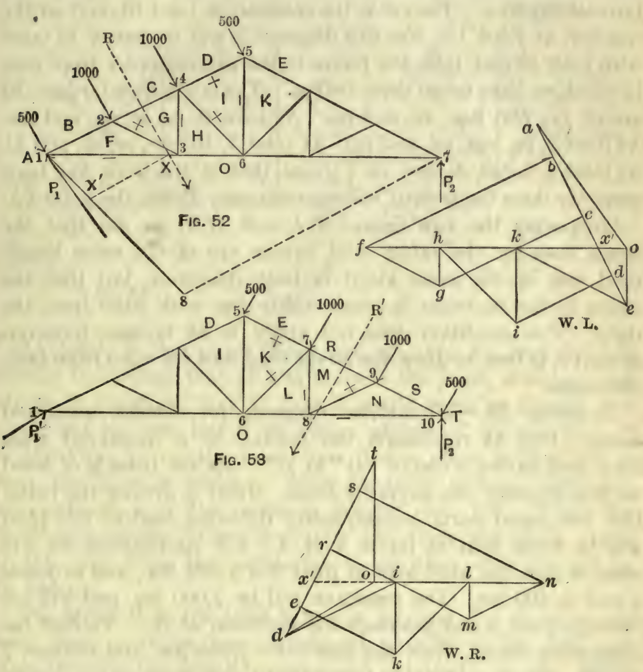


FIG. 52

FIG. 53

perpendicular to the rafter and equal to 1–8, and divided in the same proportions. Through x' on ae draw a horizontal line, and through e a vertical line, the two intersecting at o. Then eo will represent the vertical reactions at joint 7 and oa the reaction at joint 1.

The stress lines at joint 1 are: oa, $ab=500$ lbs., bf, and fo. At joint 2: fb, bc, cg, and gf. The remainder of the diagram (WL) is completed exactly as described for Fig. 51A, the only difference in the two diagrams being in the location of point o, which increases the stress in the tie-beam.

Fig. 53 represents the same truss with wind from the right. To draw the stress diagram *WR*, start with *td*, perpendicular to the rafter and equal to the total load (3,000 lbs.). Divide the line at *x′* in the same proportion as the line 1–8, Fig. 52, the longer portion being at the top. To find the reactions draw a horizontal line through *x′* and a vertical line through *t*, the two lines intersecting at *o*. Then *ot* is the reaction at joint 10, and *od* the reaction at joint 1. For this diagram it will be better to start with joint 10 and take the forces in the reverse order from that in which we have taken them before. The stress lines for joint 10 are *ot*, *ts* = 500 lbs., *sn*, and *no*. At joint 9, *ns*, *sr*, *rm*, and *mn*. At joint 8, *on*, *nm*, *ml*, and *lo*. At joint 7, *lm*, *mr*, *re*, *ek*, and *kl*. At joint 5, *ke*, *ed*, *di*, and *ik*. (Note that if the work has been correctly done the point *i* will come exactly above the point *k*.)

Comparing the two figures *WL* and *WR*, we see that the stress lines for the rafter and braces are of the same length (and also of the same kind) in both diagrams, but that the stress in the tie-beam is considerably less with wind from the right. This condition does not apply to all trusses, however, so that it is best to draw the stress diagrams for wind from both directions.

EXAMPLE 35.—Wind-stress diagram for wooden queen-rod truss. Fig. 54 represents the outline of a queen-rod truss for a roof having a rise of $14\frac{1}{2}''$ in $12''$. As the truss is of wood we will consider the supports fixed. Joint 2 divides the rafter into two equal parts, consequently the wind load at this joint will be twice that at joints 1 or 4. For convenience we will assume that the wind load at joint 2 is 1,000 lbs., and at joints 1 and 4, 500 lbs. The resultant will be 2,000 lbs. and will act through joint 2 and intersect the tie-beam at *X*. To find the supporting forces, draw the line 1–8 = 2,000 lbs. and connect 7 and 8. From *X* draw a line parallel 7–8 intersecting 1–8 at *X′*. Then *X′*–8 is the supporting force at joint 1 and 1–*X′* the supporting force at joint 7.

Begin the stress diagram (Fig. 54*A*) by drawing the line *ad* at right angles to the rafter 1–4, and equal in length to 1–8. By means of dividers locate the point *o* so that *ao* will equal *X′*–8. Then the stress lines for joint 1 will be *oa*, *ab*, *be*, and *eo*. At joint 2, *eb*, *bc*, *cf*, and *fe*. At joint 3, *oe*, *ef*, *fh*, and *ho*. At joint 4, *hf*, *fc*, *cd*, *dk*, and *kh*. It will be seen that we cannot close the figure at joint 4 without the brace *hk*, because we started at *h*, and a horizontal line through *d* will not pass through

h. Therefore a queen-rod truss requires braces in the centre panel to resist the wind stress. With the wind from the right, a brace will be required from joint 3 to joint 6.

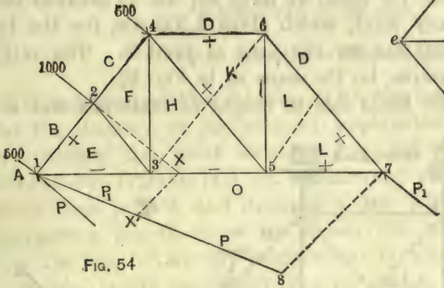


Fig. 54

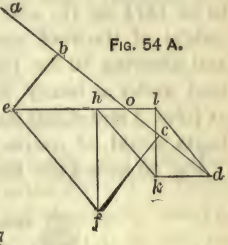


Fig. 54 A.

At joint 5 the stress lines are *oh*, *hk*, *kl*, and *lo*. It should be noticed that *lo* acts *towards* the joint, showing that *OL* is in compression. At first it would seem as though this could not be true, but if we glance at joint 7 we see that P_1 is thrusting *in* on the joint, and that a strut is required to keep the joint in position. This condition is true only when the inclination of the rafter is greater than 45°. When the inclination of the rafters is exactly 45°, there will be no stress in *OL*, and when the inclination is less than 45°, *OL* will be in tension.

The stress lines for joint 6 are *lk*, *kd*, and *dl*. If no errors have been made, a line through *d* parallel to *DL* will just pass through the point *l*, previously obtained. A very slight inaccuracy in getting the point *X'*, or in drawing the stress diagram, however, will cause the line through *d* to pass to one side or the other of point *l*, and if this happens, it shows that there has been some inaccuracy somewhere. In practice, a slight divergence will not materially affect the stresses. At joint 7 the stress polygon is *ol*, *ld*, and $do = P_1$, the lines being already drawn.

Example 36.—For the purpose of showing how the stresses due to wind and vertical loads are combined we will take the truss diagram shown by Figs. 55 and 56, being the same as shown by Fig. 12, and representing the truss shown by Fig. 3.

We will first determine the stresses due to the weight of the roof and ceiling and an allowance of 10 lbs. per square foot for sleet.

On page 954 the roof area supported at joint 2 was found to be 147½ sq. ft., and at joint 3, 200 sq. ft. On page 953 the weight of the roof was estimated at 12¾ lbs. per square foot, and allowing 10 lbs. for sleet, we have 22¾ lbs. as greatest dead load under a heavy wind, which gives 3,360 lbs. for the load at joint 2 and 4,550 lbs. for the load at joint 3. The ceiling loads would, of course, be the same as in Fig. 12.

Fig. 55 shows the loads due to weight of materials and sleet

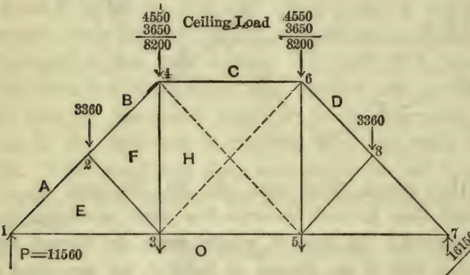


FIG. 55

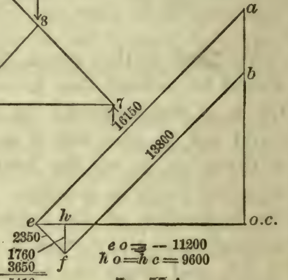


FIG. 55 A

as computed above, the ceiling loads being added to the roof loads for convenience in drawing the stress diagram. Fig. 55*A* is the stress diagram for these loads, with the stresses indicated by figures. (This diagram is drawn exactly in the same way as the stress diagram in Fig. 12.)

Wind Stresses.—The inclination of the roof is very close to 45°, and from Table VIII we find the normal wind pressure for that angle to be 27 lbs. Multiplying the roof area at joints 2 and 3 by 27, we have the wind loads indicated in Fig. 56. We must also figure the wind load at joint 1. The roof area supported at this joint, allowing 17 ins. for eave projection (see Fig. 3) is $6\frac{1}{3}' \times 15' = 95$ sq. ft., which would make the wind load 2,565 lbs. The next step will be to find the point at which the resultant of these loads would cut the rafter. As the loads

are not symmetrical or uniform on the rafter, we must obtain the point through which the resultant would act by means of moments about joint 1. The arms of the loads at joints 2 and 4 are figured on the truss diagram (Fig. 56). The moments are

$$3{,}990 \times 9\tfrac{5}{12} = 37{,}572$$
$$5{,}400 \times 18\tfrac{1}{4} = 98{,}550$$
$$\text{Total moment} = 136{,}122 \text{ ft.-lbs.}$$

The resultant will be the sum of all the loads, or 11,955 lbs., and the distance of its point of application from 1 is found by dividing the total moment by the resultant, 136,122 divided by 11,955 = 11.4 ft. Measuring off 11.4 ft. on the rafter from joint 1 and drawing a line at right angles to it intersecting the tie-beam we obtain the point X. From 1 draw the line 1–8 at any angle equal in length to the sum of the loads, 11,955 lbs., and connect 7 and 8. From X draw a line parallel to 7–8, intersecting 1–8 at X'. Then X'–8 will be the supporting force at joint 1.

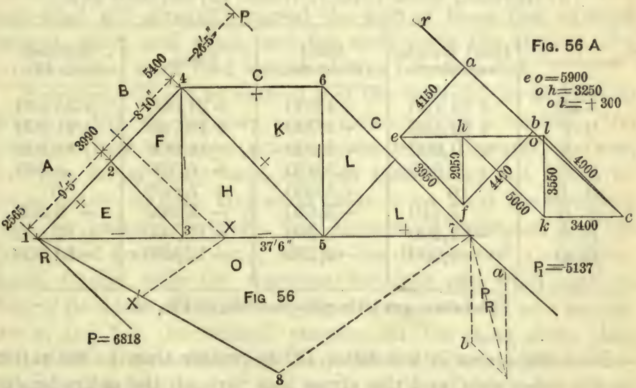


FIG. 56 A

FIG 56

Supporting Forces Computed by Moments.—The supporting forces can also be computed by moments. The moments of the loads about joint 1 tend to rotate the truss from left to right. To prevent this rotation we have the supporting force P_1 acting at joint 7. To just maintain equilibrium, the moment of P_1 about joint 1 must just equal the moments of the loads about the same point, which we found above to be

136,122 ft. lbs. The arm of P_1 is the perpendicular distance between its line of action and joint 1. Continuing P_1 above the truss we obtain the dotted line at p, and the distance from 1 to p is 26.5 ft.

Knowing the arm, the value of P_1 is obtained by dividing the moments of the loads 136,122 by the arm, or 26.5 ft, which gives 5,137 lbs. As the sum of P and P_1 must equal the total load, $P=11,955-5,137=6,818$. The distance 1–X' and X'–8 should scale reasonably close to these figures. Knowing the supporting forces, the stress diagram, Fig. 56*A*, is drawn exactly as described for Fig. 54*A*. As the inclination of the rafters is a little greater than 45°, *OL* is in compression, but the stress is very small. The figures on Fig. 56*A* indicate the stresses in pounds. We are now prepared to tabulate the stresses, which should be done as in the following table.

In tabulating the wind stresses, it should be remembered that the wind may blow against either side of the truss, and the greatest stress liable to occur should be put in the table.

STRESSES FOR TRUSS (Figs. 12, 55, and 56).

Member.*	Dead Weight and Sleet.	Wind Stresses.	Total.	Stresses (Fig. 12).
A–E	+16,150	+4,900	+21,050	+25,640
B–F	+13,800	+4,900	+18,700	+21,400
C–K	+ 9,600	+3,400	+13,000	+14,900
E–F	+ 2,350	+3,950	+ 6,300	4,400
H–K	0	+5,000	+ 5,000	0
F–H	− 5,410	−3,550	− 8,960	− 6,900
E–O	−11,200	−5,900	−17,100	−17,800
H–O	− 9,600	−3,250	−12,850	−14,900

* Members are lettered according to Fig. 56.

Thus the stress in the rafter *LC* is greater than in the rafter on the other side, and this stress acts through the entire length of the rafter; hence the stress for *AE* and *BF* should be entered as 4,900 lbs. (the stress in *LC*). In the same way the stress in the rod *KL* is greater than in *FH*; hence the stress in *KL* should be tabulated. The stress in *LO* would slightly reduce the tension due to dead load, but as the stress in *EO* increases it, the stresses in *EO* and *HO* should be tabulated.

Both sides of the truss should of course be made alike, and

two braces should be inserted in the centre panel. In the fifth column of the table we have given the stresses due to the ceiling load and a vertical load on the roof of $42\frac{3}{4}$ lbs. per square foot, as obtained from the stress diagram, Fig. 12. Comparing the stresses in the fourth and fifth columns, we see that except for the brace *EF*, and for the two rods, the stresses obtained by combining snow and wind and adding to the dead weight are greater than the totals due to wind, dead weight, and sleet. Vertical loads, of course, give no stress for the braces in centre panel, and unless the wind stresses are drawn, it will be necessary to guess at the sizes of these braces. The stress in these braces, however, is so small that it will not require a very large piece of timber.

The stresses given in the fourth column are unquestionably nearer what the real stresses are likely to be than those in the fifth column. If the roof were to be erected in a warm climate where there would be no sleet, then these stresses could be further reduced by omitting the 10 lbs. per square foot added for sleet.

If, on the other hand, the inclination of the roof was less than 30°, the stresses produced by a heavy fall of snow without wind will generally exceed the sum of those due to dead weight, sleet, and wind, and for such roofs the stresses due to maximum snow load should always be computed.

Reactions.—It will be noticed that the reactions, or supporting forces in Fig. 56, are very much inclined from a vertical. As the dead load is always acting on the truss, however, the real reaction would never have such an inclination, but would be more nearly vertical, and when there is no wind the reactions will be exactly vertical. The theoretical reaction, due to both wind and dead load, will be the diagonal of a parallelogram formed with the reactions for dead and wind loads as two of its sides. Thus if 7–*a*, Fig. 56, represents P_1 at a smaller scale, and 7–*b* the vertical reaction to the same scale, then P_R will be the resultant reaction, which will be modified somewhat by friction.

Examples 33, 34, and 35 serve to show the general method of drawing wind-stress diagrams, and are sufficient to enable the student to draw those diagrams for most trusses with straight rafters. For trusses with curved rafters the diagrams become more complicated, and the reader is referred to "Graphical Analysis of Roof Trusses," by Prof. Charles E. Greene,

also to "Steel Mill Buildings," by Prof. Milo S. Ketchum, for explanations regarding wind-stress diagrams for such trusses. The latter work also takes up in detail stress diagrams for trusses supported and braced from steel columns, and will be found of valuable assistance in designing steel mill buildings.

CHAPTER XXVII.

ROOF-TRUSSES (*Continued*).

Proportioning the Members and Detailing the Joints.

Proportioning the Members to the Stresses.—The size or sectional area of the different members of a truss cannot be proportioned with any accuracy until the stresses which the maximum loads will produce have been found. When the stresses are known, however, the size of the truss members can be readily computed by the rules and tables for the strength of materials.

Every member of a truss must be either a *tie* or a *strut*. If either is also subject to a transverse load, as is often the case with rafters and tie-beams, then the member becomes a *tie-beam* or *strut-beam* according as it is in tension or compression. We therefore have four kinds of members (regardless of material) to be considered in trusses, viz., simple ties, simple struts; tie-beams and strut-beams.

Rules and data for computing the sectional area of simple ties of any shape or material are given in Chapter XI. Struts may be computed by the rules and tables given in Chapter XIV, and directions for proportioning steel tie- and strut-beams to the load and stress are given on pages 511 and 512, and for wooden strut- and tie-beams on pages 568 and 569.

To more fully show the application of the rules, however, we will compute the size of members for three or four trusses.

EXAMPLE 1.—To compute the size of the members in the truss shown by Fig. 1.

This is the same truss as is represented by Figs. 3, 12, 55, and 56 of Chapter XXVI.

For the stresses we will use those given in the fourth column of the table on p. 1034, and which are tabulated below. The members *RR* are to be wrought-iron rods, not upset, and all other members are to be of a good quality of white pine.

None of the members of this truss are subject to a transverse strain, so that we have to consider only simple ties and simple struts.

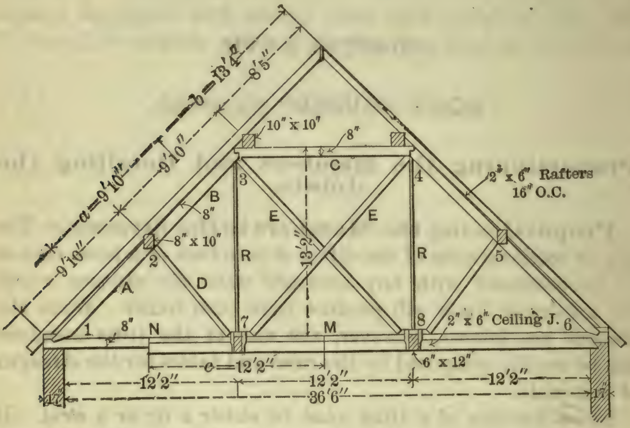


Fig. I

Rods.—The tension in each of the two rods (see table below) is 8,960 lbs. As a considerable portion of the stress is due to wind-pressure we can safely use a unit stress of 12,500 lbs. to the square inch. From the table on p. 340 we find that the safe strength of a $1\frac{1}{8}''$ rod, not upset, at 12,500 lbs. to the square inch is 8,570 lbs., and of a $1\frac{1}{4}''$ rod 11,060 lbs. The $1\frac{1}{8}''$ rod is not quite strong enough and we will use a $1\frac{1}{4}''$ rod, or we might use a $1\frac{1}{8}''$ steel rod.

STRESSES AND DIMENSIONS FOR TRUSS I.

Member.	Stress, Lbs.	Dimensions.
A	+21,050	6×8, white pine
B	+18,700	6×8, white pine
C	+13,000	6×8, white pine
D	+ 6,300	4×6, white pine
E	+ 5,000	4×6, white pine
N	−17,100	3 2″×8″ ¾″ bolts, 2′ centre to centre
M	−12,850	3 2″×8″ ¾″ bolts, 2′ centre to centre
R	− 8,960	1 ¼″ rod, wrought iron, not upset

Rafter.—The stress in the rafter is 21,050 lbs. at A and 18,700 lbs. at B, but as it should be of the same size for its entire length we will proportion it to the stress at A. The clear length between joints is about 9 ft. From the table for white-pine posts, p. 412, we see that a 6×8 timber is more than strong enough, while a 6×6 is hardly sufficient. We will therefore make the rafter 6×8.

Strut-beam (C).—The stress in this member is 13,000 lbs. and it is about 12 ft. long between bearings. From the same table, p. 412, we see that a 6×6 timber would do for this member, but on account of making a better joint we will make it of the same size as the rafter, or 6×8.

Braces.—The stress in the brace D is 6,300 lbs. From the table, p. 412, we see that a 4×6 timber will have ample strength. The stress in the braces EE is 5,000 lbs. and the length about 17 ft. As the braces stiffen each other at the intersection, however, we can safely make them $4'' \times 6''$, notching each brace 1 in. where they pass, so that the total thickness at the intersection will be $6''$, and bolting them together with a $\frac{1}{2}''$ bolt in a $\frac{3}{4}''$ hole to give a little play. Brace D had better be set flatways and the braces E edgeways.

Tie-beam.—The maximum tension in the tie-beam is 12,850 lbs. The tensile strength of white pine is given on p. 322 at 1,400 lbs. per square inch, therefore the tie-beam must have a net sectional area at $N = 12{,}850 \div 1{,}400$ or 9.2 sq. ins. A $2'' \times 6''$ plank if continuous from end to end of the truss and without holes would therefore resist the stress, but to make good joints with the rafters and braces, the tie-beam must be as wide as the rafters.

We must also allow for the holes where the rods pass through and for a slight notch at joints 7 and 8. A 6×6 timber in one length will have ample strength, but as this may be difficult to obtain, we can build it up of three $2'' \times 8''$ planks bolted together, using 24-ft. and 14-ft. lengths and lapping the long pieces so that there will be 10 ft. between the joints.

The centre planks will be cut so nearly in two by the rods that they will have little strength, and the stress in the centre will practically have to be borne by one plank.

The planks should be bolted together by $\frac{3}{4}''$ bolts 2 ft. on centres. We now have the dimensions for all members of the truss, and they should be entered in the table opposite the stresses.

Note. In this example we have an excess of strength in the

timber, but as none of the timbers are large, nothing to speak of would be gained by trying to cut them down. When the stresses are four or five times as great as in this truss, requiring large timbers, then the size may be figured more closely, as the cutting at the joints is not as much in proportion to the sectional area in large timbers as in small ones.

EXAMPLE 2.—For this example we will take the truss shown by Fig. 2, which is the same truss as is shown by Figs. 4 and 24 of Chapter XXVI. The stresses produced by a vertical dead load of 42½ lbs. per square foot of roof for wind and snow are figured on the stress diagram, Fig. 24*A*, Chapter XXVI, and reproduced in the following table.

The rafters and tie-beams of this truss also sustain a transverse load which is uniformly distributed. In computing the joint loads for this truss (see Example 3, Chapter XXVI), we allowed 12 lbs. per square foot for the weight of the ceiling, which will give the transverse loads for *S* and *B* indicated in the following table.

The transverse loads for the rafters *A* and *B* are figured at 42½ lbs. per square foot.

STRESSES AND DIMENSIONS FOR TRUSS 2.

Member.	Stress, lbs.	Dimensions, Material White Pine.
A	Compression, 8,000 Transverse, 1,000	2 1¾″×8″
B	Compression, 6,600 Transverse, 1,320	2 1¾″×8″
D	Compression, 1,890	1 2×8
E	Compression, 750	1 2×8 or 2×10
F	Tension, 4,350	2 1×8
H	Tension, 2,530	2 1×6
S	Tension, 1,875 Transverse, 470	1 2×10
T	Tension, 5,400 Transverse, 384	1 2×8
T_1	Tension, 1,875	1 2×8

We will assume that the truss is to be built of a good quality of white pine, spiked and bolted at the joints.

Rafter.—The compression in the bottom of the rafter *A* is greater than at *B*, but as the length of *B* is considerably greater than *A*, and the transverse load is also greater on *B*

if the rafter is strong enough to resist the strains at B, it will be strong enough to resist those at A.

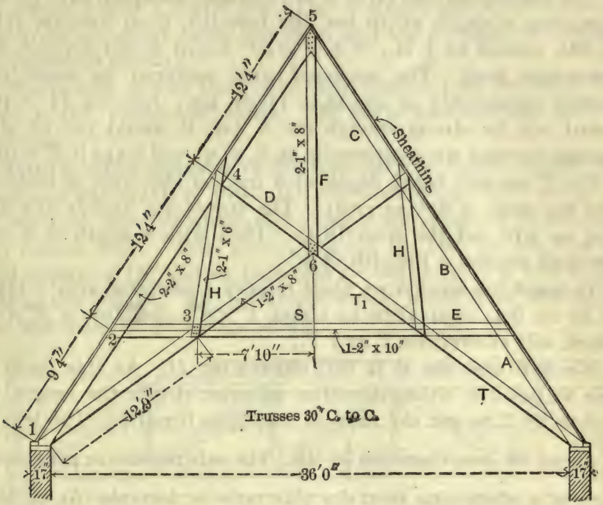


Fig. 2

We will first find the dimensions necessary to resist the transverse load. As the rafter is inclined its strength as a beam is considerably greater than if horizontal; we therefore will use for the span 9 ft., which is about an average of the real length and the horizontal projection.

To support 1,320 lbs. with a span of 9 ft. will require a $1\frac{5}{8}'' \times 8''$ joist (formula 11, p. 564). Considering the resistance to compression, it should be remembered that the sheathing is nailed to the rafters, and hence they cannot bend sideways. The depth of the rafter can therefore be considered as the breadth, so that the ratio of length to breadth will be $\frac{144}{8} = 18$ (the clear distance between the joints should be taken for the length as a strut).

By formula (5), p. 410, we find that the safe resistance for a white-pine strut with a ratio of length to breadth of 18 is 517 lbs. per square inch, hence it will require 13 sq. ins. to resist 6,600 lbs. This is equivalent to $1\frac{5}{8} \times 8$. Therefore two pieces each $1\frac{5}{8}'' \times 8''$ will be strong enough for the rafters.

Tie-beams.—These should preferably be of single planks spiked between the rafters.

The transverse load on S is 470 lbs. and the span, say, 15′ 6″. Assuming a depth of 10 ins. the breadth, from formula (11), p. 564, should be $\frac{3}{4}$ in., or a $\frac{3}{4}'' \times 10''$ board would support the transverse load. The sectional area required to resist the tension equals $\frac{1875}{1400}$ or less than $1\frac{1}{2}$ sq. ins.; hence a $1\frac{1}{8}'' \times 10''$ board will be strong enough for S, but it would not be stiff enough to resist the compression at E, so we will make it $2'' \times 10''$.

For T we will try a depth of 8 ins., as the piece is inclined and the span is not as great. The transverse load is 384 lbs. and we will call the span 12 ft. Then with a depth of 8 ins. we shall require a breadth of $\frac{5}{8}''$.

To resist the tension we shall require a sectional area $=\frac{5400}{1400}$, or $3\frac{1}{7}$ sq. ins., equivalent to about $\frac{1}{2}'' \times 8''$; therefore a $2'' \times 8''$ plank will answer for T and T_1.

We will now see if it will answer for D. As this piece is free to bend in either direction we must divide the length, in inches, by 2, to get the ratio of length to breadth. The length is about 86 ins., therefore $\frac{l}{b} = 43$. The safe resistance per square inch of a white-pine strut for this ratio is, formula (5), p. 410, 367 lbs.; hence we shall require a little more than 5 sq. ins. to resist 1,890 lbs. A 2×8 strut is therefore ample.

Ties F and H.—For F we shall require a sectional area $=\frac{4350}{1400}$, or $3\frac{1}{8}$ sq. ins., and for H $\frac{2530}{1400}$, or 1.8 sq. ins. As it would be impossible to secure the ends of such small pieces, we will make the tie F of two 1×8's and HH of two 1×6's, so as to have plenty of room for nailing.

(*Note.* Wooden ties must always be much larger than the sectional area required by the formula for tension, to provide means for making a satisfactory joint.) We have now computed the dimensions for all parts of the truss, as indicated in the table. By continuing the boards F to the tie S, so as to shorten the span, we could reduce S to $2'' \times 8''$.

EXAMPLE 3.—For this example we will take the Howe truss, shown by Fig. 3. The trusses are supposed to be uniformly spaced 15′ 8″ centre to centre and the rafters and ceiling joists span from truss to truss and rest directly on the chords.

Allowing 30 lbs. for snow and 6 lbs. for tar and gravel roofing, the dead load per square foot of roof will be $46\frac{1}{2}$ lbs., and for the ceiling we will allow 16 lbs. per square foot. This would

make the roof load 5,885 lbs. at joint 2 and 5,704 lbs. at joints 4 and 6. The ceiling load at joint 3 would be 1,984 lbs. and at joints 5 and 7 1,968 lbs. Fig. 4 shows the dimensions of the truss on centre lines, the loads and the stresses that would be produced thereby.

The figures above the chord lines preceded by the letter *W* denote the transverse loads. All loads and stresses are in pounds.

Rods.—The diameter of the rods may be found directly by means of Table III, p. 340. We will assume that they are to be of wrought iron, not upset, and allow a unit stress of 12,500 lbs. per square inch. Then from the table (p. 140) we see that it will require a $1\frac{3}{8}''$ rod at joint 2, a $1''$ rod at joint 4, and a $\frac{9}{16}''$ rod at the centre. As the last, however, would look very light we will make it $\frac{3}{4}''$ in diameter.

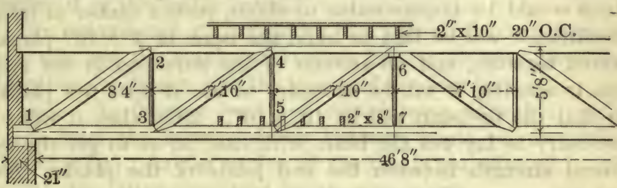


Fig. 3

Top Chord.—The load on one of the centre spans of the top chord is 5,704 lbs. and the span is about $7\frac{3}{4}$ ft. Assuming a depth of 10 ins. we find the breadth required to resist this load (by formula (11), p. 564, the wood being white pine)

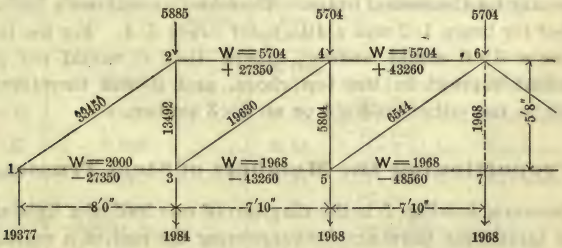


Fig. 4

=3.7 ins. The compression in the centre panels is 43,260 lbs. From Table V, p. 412, we see that a $10''\times 10''$ post 8 ft. long will support 62,500 lbs. Therefore to support 43,260 lbs. will re-

quire a 7″×10″ timber. Adding together the thickness required for the transverse load and for the compressive stress, we have 10.7″×10″ as the required size, which will require using a 12″×10″ timber. As the timber will be stronger if placed vertically, we will make the top chord 10″×12″, and build it of five 2″×12″ planks, bolted together. Between the end joints and the wall the chord can be reduced to 6″×12″.

Tie-beam.—The transverse load on centre panel is 1,968 lbs. Assuming a depth of 10 ins., it will require a thickness of 1¼″ to sustain the transverse load. To resist the direct tension will require a net sectional area $=\frac{48,560}{1,400}=35$ sq. ins., or 3½″×10″. Between the joints this must be increased by 1¼″ to resist the transverse strain, so that a beam 4¾″×10″ is the least that would answer for a solid tie-beam, i.e., a single stick of timber. As it would be impracticable to obtain such a timber in most localities, it will be best to build the beam of 2″×10″ planks bolted together, and on account of the reduction in net area due to splicing, it will be necessary to use at least five planks so that the tie-beam will be 10″×10″. Even then it will be necessary to lay out the beam with care, so as to get the required strength between the end joints of the planks. See p. 1058.

Braces.—As the chords are to be 10 ins. wide, the braces 1–2 and 3–4 should be of the same width. Brace 5–6 may be reduced to 8 ins. wide. The length of the braces is about 9 ft. From Table V, p. 412, we see that a 6×10 is not quite strong enough for the outer brace, and is a little stronger than necessary for the second brace. Therefore we will use a 10″×8″ timber for brace 1–2 and a 10×6 for brace 3–4. For the inner braces a 3×6 would answer, except that it would not give sufficient support to the top chord, and it will therefore be better to use either an 8×4 or an 8×3 timber.

Proportioning the Members of Steel Trusses.

Example 4.—Fig. 5 is the diagram of one half of a light steel truss built some years ago for supporting the roof of a machine-shop in New York State. The trusses were spaced 8 ft. centre to centre. The roof consisted of 3″ plank, spiked to a 3×6 bolted to the rafters of the truss, and covered with a patent roofing similar to ruberoid. The actual weight of the roof

is therefore very small, but as it is quite steep it would be well to allow 40 lbs. per square foot of roof surface for obtaining the stresses. As the roof area supported at each joint is 49⅓ sq. ft., this would make the panel loads 1,973 lbs., or say 2,000 lbs. The stresses due to these loads are indicated on the diagram, Fig. 5. For a roof construction such as this, it is more economical to divide the rafter into uniform panel lengths, as in Fig. 5. When this is done, the stresses in the four web-braces, as *BD* and *GL*, are equal, also in the two ties *DE* and *EL*, so that it is only necessary to compute the size for one strut and one tie.

As the stresses in the rafters are comparatively small, it will also be more economical to make the rafter of the same section for its entire length.

The tie-beam we will make of one section from *A* to *F*, and reduce it in the center panel.

For convenience in computing the size of the members we will tabulate the maximum stresses for the different members, omitting duplicates, also the length of the members.

(*Note.* In figuring length of struts, some reduction can be made from the exact distance between joints, on account of the riveting.) As the section of the ties is not affected by the length, the latter is omitted for those members.

STRESSES AND SIZES (Fig. 5).

Member.	Stress, Lbs.	Ap. Length, Inches.	Net Area of Section Required.*	Section Selected.
			Sq. Ins.	
A–F	−16,900	....	1.13	2, $2\frac{1}{2}\times 2\times\frac{1}{4}$ L's. Net area 1.76 sq. ins.
F–M	− 9,200	....	0.55	2, $2\times 2\times\frac{1}{4}$ L's. Net area 1.52 sq. ins.
F–K	− 7,700	....	0.52	1, $2\times 2\times\frac{1}{4}$ L. Net area 0.76 sq. in.
D–E	− 3,070	....	0.21	1, $2\times 2\times\frac{1}{4}$ L. Net area 0.76 sq. in.
A–K	+20,200 2,000 t. l.	72	....	2, $2\times 2\times\frac{1}{4}$ L's. 1 $10''\times\frac{1}{4}''$ web-plate. $r=3.1$
B–D	+ 1,900	72	....	1, $2\times 2\times\frac{1}{4}$ L. $r=0.39$. Safe strength 4,000 lbs.
E–F	+ 5,000	144	....	2, $2\times 2\times\frac{1}{4}$ L's. Cross-section. $r=0.94$. Safe stgth, 10,980 lbs.

* At 15,000 lbs. per square inch.

Dimensions of Tension Members.—For these members we can safely use a working strength of 15,000 lbs. per square inch of net section. Dividing the stresses by 15,000 lbs., we obtain the required net sectional areas given in the fourth column of our table. We must next select angles having a sectional area slightly in excess of these figures.

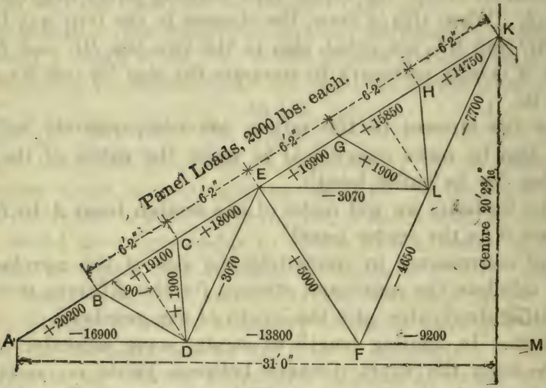


Fig. 5

For the main tie, AF, it is necessary to use two angles in order to make satisfactory joints. For AF, then, we will use two angles each having a sectional area slightly in excess of $\frac{1.13}{2}$, or .57 sq. in. The sectional areas for angles of all sizes are given on pp. 302–311. On p. 311 we find that a $2'' \times 2'' \times \frac{3}{16}''$ angle has a sectional area of .72, which would be large enough for our purpose, but it is a good rule not to use anything less than $\frac{1}{4}''$ in thickness, and for the principal tie of a truss two $2\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$ angles are about the least that should be used; therefore we will use that joint for the tie from A to F. From F to the corresponding joint on the other side we will use two $2'' \times 2'' \times \frac{1}{4}''$ angles.

For the ties FK, DE, and EL we will use single $2'' \times 2'' \times \frac{1}{4}''$ angles. (*Note.* A $2'' \times 2''$ angle is the smallest size that it is practicable to use in roof-trusses, although they are frequently used only $\frac{3}{16}''$ thick.)

The sizes of angles selected and the net sectional areas should be put in the table. The net sectional area is obtained by

subtracting from the area given in the tables the amount cut out by one rivet-hole, which may be obtained from Table I, on p. 640. For this truss, as the members are all small, we will use $\frac{5}{8}''$ rivets.

Compression Members.—For the main rafter we will use two angles and a web-plate, as in Fig. 6, as this is an economical section for a strut-beam, and a good section for making joint connections.

We will proportion the section so that the plate will be capable of resisting the transverse strength and the angles the compressive stress.

We will assume a depth for the plate of 10 ins. and find the necessary thickness. This may be computed by the same formula as given for wooden joists, formula (11), p. 564, using 888 lbs. for A. The transverse load on each panel is 2,000 lbs. Then $t=\frac{6' \times 2{,}000}{2 \times 100 \times 888}=.07$ in., or less than $\frac{1}{10}$; but it would not do to use a plate less than $\frac{1}{4}$ in. thick, therefore we will make the web-plate $10'' \times \frac{1}{4}''$.

To find the size of angles required to resist the compressive stress, we must assume some size, then find the radius of gyration of the entire section, and then the strength as a strut.

To find the radius of gyration, we must first find the centre of gravity of the section, then the moment of inertia, and finally the radius of gyration.

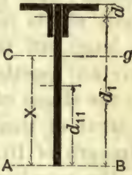


Fig. 6

The distance X of centre of gravity from axis AB (see p. 240)

$$=\frac{\text{area of plate} \times d'' + \text{area of both angles} \times d'}{\text{area of entire section}}.$$

As the stress in the rafter is comparatively small we will try $2'' \times 2'' \times \frac{1}{4}''$ angles, having a total area of 1.88 sq. ins.

The distance d we find from the table, p. 311, to be .59, so that $d'=10-.59=9.41''$. d'', of course, $=5''$. Then $x=\frac{2.5 \times 5+1.88 \times 9.41}{2.5+1.88}=6.9$. The moment of inertia of the section about $C-g$ is found by the rule on p. 282, as follows:

$$\text{Moment for plate} \begin{cases} \frac{bd^3}{12}=20.8 \\ 2.5 \times 1.9^2=9.025 \end{cases}$$

$$\text{Moment for angles}\left\{\begin{array}{ll} 2\times\ .35\ (\text{p. } 311) & =\ \ .70 \\ 1.88\times 2.51^2 & =11.84 \end{array}\right.$$

$$\text{Moment for entire section} \qquad =42.36$$

The radius of gyration is found by dividing the moment of inertia by the area of the section and taking the square root of the quotient (see p. 289). 42.36 divided by the area of the section$=9.67$, and the square root of this is $3.1=r$. As the length of each section of rafter may be taken as 6 ft., $\frac{l}{r}=\frac{72}{3.1}=23.2$. From Table XI, p. 463, we see that when $l \div r$ is less than 30, we should use 12,000 lbs. per square inch in computing the strength of the strut. Multiplying the area of the two angles (1.88) by 12,000, we have 22,560 lbs. as the safe resistance to compression, and as this is in excess of the stress, we will use this section for the entire length of the rafter.

(*Note.* The above process for finding size of rafter seems long and tedious, but there is no shorter way of finding the strength of a strut of this section with any degree of accuracy.)

The other struts, being simply in compression, can be proportioned directly by means of the tables in Chapter XIV. Thus for the strut *BD*, which is 72 ins. long, we find from the table, p. 469, that a single $2\times 2\times \frac{1}{4}$-in. angle has a strength greater than the stress, and we will therefore make the four short struts of single 2×2 angles. For the strut *EF*, which is long, but not very heavily strained, we will use two angles riveted together by plates, so that the cross-section will be in the shape of a cross. At the bottom of p. 470, we find that a strut of this section formed of two $2\times 2\times \frac{1}{4}$-in. angles has a strength of 10,980 lbs. for a length of 12 ft., or twice the stress in the member; therefore we will use that section. For members in compression it is not customary to deduct for rivet-holes. We have now determined the sizes for all members of the truss, as indicated in the table.

Example 5.—Fig. 7 is the diagram of one half of a steel truss designed and built by the Berlin Iron Bridge Co. for the Elerslie Coal and Coke Co., at Winifrede Junction, W. Va. The roof is covered with slates fastened to angle-iron purlins running from truss to truss and spaced $10\frac{1}{2}$ ins. c. to c. The stresses indicated on the diagram are those that would be due to a vertical load of 34 lbs. per square foot of roof surface, with trusses spaced 8 ft. on centres. The author does not know

for what loads the truss was actually computed, but they were probably somewhat in excess of those assumed. In the truss as actually built all of the members are formed of pairs of angles, except ties *CD* and *DF*, which are single angles.

To facilitate determining the size of angles to be used for the different members, the stresses and length of struts are given in the following table. The lengths, however, are not the actual lengths, but are what may be termed the distance between centres of bearings, considering each member as ending at the joints.

The loads marked *t. l.* are the transverse loads on the rafter.

DATA FOR PROPORTIONING MEMBERS OF TRUSS (Fig. 7).

Member.	Stress.	Length.	Net Sectional Area Required.*	Angles Selected.	Net Sec.
			Sq. In.		Sq. In.
AC	−21,800	......	1.46	2, 3 $\times 2\frac{1}{2} \times \frac{1}{4}$	2.18
CE	−18,700	......	1.25	2, $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	1.94
EH	−12,500	......	0.84	2, $2\frac{1}{2} \times 2 \times \frac{1}{4}$	1.68
EF	− 6,250	......	0.42	2, 2 $\times 2 \times \frac{1}{4}$	1.44
FG	− 9,300	......	0.62	2, 2 $\times 2 \times \frac{1}{4}$	1.44
CD	− 3,050	......	0.21	1, 2 $\times 2 \times \frac{3}{16}$	0.56
AD	+23,500 2,320 t. l.	8′ 6″	......	2, 5 $\times 3\frac{1}{2} \times \frac{3}{8}$	
DG	+21,600 2,320 t. l.	8′ 6″	......	2, 5 $\times 3\frac{1}{2} \times \frac{5}{16}$	
BC	+ 2,325	3′ 0″	......	2, 2 $\times 2 \times \frac{1}{4}$	
DE	+ 4,650	6′ 6″	......	2, 2 $\times 2 \times \frac{1}{4}$	

* At 15,000 lbs. per sq. inch.

Tension Members.—First find the net sectional areas required for these members by dividing the stresses by 15,000 lbs., the allowed stress per square inch, and put them in the fourth column. Then, by means of the tables on pp. 306–311, select the angles having sectional areas a little in excess of the required area and put the sizes in the fifth column. It is also a good idea to put down the net areas of the angles selected for the tension members, which are obtained by deducting from the actual area the allowance for one $\frac{3}{4}$-inch rivet, p. 640.

Rafter.—As the rafter has a transverse load, it will be

necessary to assume some section and then compute its strength as a strut-beam, and if its strength is not equal to the combined stress and load, we must try a larger section.

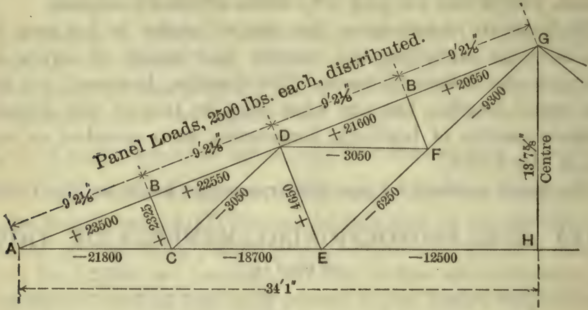


Fig. 7

For the rafter from A to D we will try two $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles. From the table on p. 529 we find the coefficient for one angle to be 12.21 tons, or 24,420 lbs., and dividing by the span 8.5 ft., we have 2,873 lbs. as the safe transverse load, with long leg vertical, and two angles would support 5,746 lbs. The actual load is 2,320 lbs., or 44 per cent. of the strength of the angles.

From the table on p. 470 we find the strength as a strut of two angles of this size, 8′ 6″ long, to be about 30.9 tons, or 61,800 lbs. The actual stress is 23,500 lbs., or only 38 per cent. of the strength of the section. As it requires 44 per cent. to resist the transverse load, it will require 82 per cent. to resist both, and as a smaller section would probably not be strong enough we will use two $5 \times 3\frac{1}{2} \times \frac{3}{8}''$ angles for the rafter from A to D. For the upper half of the rafter we will try the same size of angles with a thickness of $\frac{5}{16}''$. The coefficient for this thickness is (p. 530) 10.34 tons, or 20,680 lbs., and dividing by the span we have 2,433 lbs. as the safe strength of one angle. Therefore it will require about 50 per cent. of the strength of two angles to support the actual load.

The resistance to compression of two $5 \times 3\frac{1}{2} \times \frac{5}{16}''$ angles is not given in the table, p. 470, but we find the difference between the loads given for $\frac{3}{8}$ and $\frac{3}{4}$ thicknesses to be for 8 ft. length 28.96 tons, which would be 4.83 tons for each $\frac{1}{16}$ in. in thickness, so if we subtract 5 tons from the safe load for

the $\frac{3}{8}$-in. thickness, we will have the safe load for $\frac{5}{16}$ in. Making the subtraction we have 26.66 tons as the safe load for length of 8 ft., and to obtain the strength for 8′ 6″, we should subtract about .62 ton, which would give the safe strength, say 26 tons, or 52,000 lbs., for a length of 8′ 6″. As the stress is only 21,600 lbs., we shall utilize only 42 per cent. of the strength of the strut, and as we require 50 per cent. to resist the transverse load, we require 92 per cent. in all; therefore this section is strong enough for the upper portion of the rafter.

Struts.—These we can find directly from the table on p. 470. For all three of the struts we will use two $2 \times 2 \times \frac{1}{4}''$ angles, which, while they have considerable excess strength, are as light as should be used.

The actual size of angles used in this truss are indicated on Fig. 28.

Joints of Wooden Trusses.

It is probably safe to say that the joints in wooden trusses, taking them as they are found throughout the States, are the weakest portion of the truss, and especially the joints at the ends of wooden ties. For example, a $6'' \times 6''$ timber of Georgia pine would require a force of 288,000 lbs. to pull it apart, but it is practically impossible to secure the end of such a timber so as to develop its full tensile strength. The splicing of tie-beams also is often a weak place in many trusses.

The joints of any truss should be proportioned with as much care as the size of the members, so that the truss will be equally strong in all its parts. The principles by which the strength of joints on which a pulling stress is exerted are explained at length on pp. 382–397, and illustrated by a few examples. To explain the subject still further, we will show how the joints of the trusses illustrated by Figs. 1 and 3 (of this chapter) should be made.

The first and most important joint of the truss shown by Fig. 1 is joint 1, where the truss rests on the wall. There are several ways in which this joint may be made, the simplest being a bolt joint like that shown by Fig. 8. In trusses having a horizontal wooden tie-beam, the tie-beam almost invariably extends over the support, and the rafter or principal strut bears on top of it. The correct method of properly portioning a joint such as is shown by Fig 8 is explained on pp 392–395.

In this case the thrust in the rafter is 21,050 lbs., and to find the theoretical stress in the bolt, we should first make a drawing of the joint at a scale of about 1 in. to the foot, giving the rafter its correct inclination, and locating it on the tie-beam, so that the point where the central lines of the tie-beam and rafter intersect will be at least 6 ins. in on the wall. Then draw the

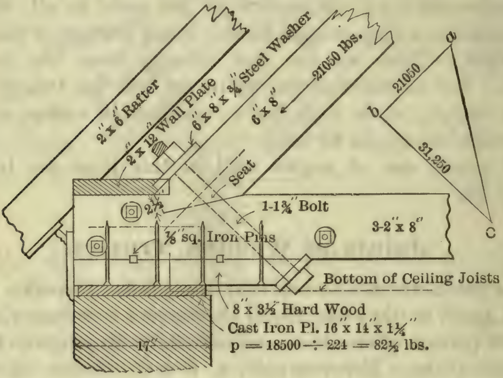


Fig. 8
Joint 1 of Fig. 1.

notch so that the toe of the rafter will be about 2½ ins. deep, and a little to one side draw a line *ab* parallel and equal to the stress in the rafter, to a scale of pounds, and from the upper end *a* a line at right angles to the seat of the rafter, and from the lower end *b* a line at right angles to the rafter, and parallel to the bolt. Then the line *bc*, measured by the scale with which *ab* is drawn, gives the stress in the bolt. In this case the line *bc* scales 31,250 lbs. To find the diameter of the bolt necessary to resist this stress we should use table IX, p. 384. From that table we find that to resist 31,250 lbs. will require a 1¾″ bolt. The bolt should be placed at right angles to the rafter, and should have a good-sized washer at each end (see Washers, p. 1064). It will not do to cut into the tie-beam sufficient to get a proper bearing for so large a bolt, therefore we must either put a wooden block under the truss, as in Fig. 8, or use a cast-iron washer, as in Fig 33, p 394 For light trusses the wooden block answers the purpose as well as the cast washer and will generally be cheaper. To prevent the block from sliding, notches should

be cut in the top of the block and the bottom of the tie-beam and 1″ or $\frac{7}{8}$″ square iron bars driven in. The bolster should also be well spiked to the tie-beam before the bolt is put in place.

As a rule, it is a good idea to place the wall plate which receives the common rafters just above the tie-beam of the truss, the wall being built around the truss. This affords an opportunity for getting at the nut on the bolt to tighten it in case the wood shrinks.

The *bearing* of a truss on the wall should always be considered, and a plate or heavy stone provided which will reduce the pressure to within the limits given on page 399. In this case we will use a 16″×14″×1$\frac{1}{4}$″ cast-iron plate, which reduces the pressure on the brickwork to 82$\frac{1}{2}$ lbs. per square inch.

Joint 1 of Fig. 3.—This joint might be made in the manner shown by Fig. 35, p. 396, but if the tie-beam is to be cased, the projection of the cast-iron washers below the tie-beam is objectionable. Fig. 9 shows another method of making

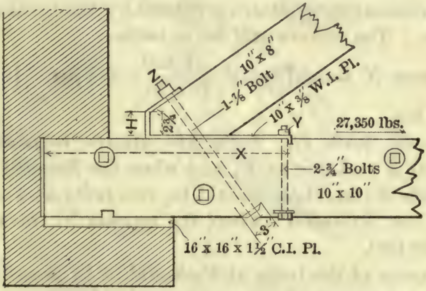


Fig. 9

Detail of Joint 1, Fig. 3.

the bearing joint of a wooden truss which avoids the use of large bolts and projecting bolt-heads. This is a strong joint, especially serviceable for heavy stresses, and where the inclination of the rafter is less than 45 degrees. The points to be computed in this truss are: Area of bent plate, height at toe, and the distance X. The sectional area of the plate (which should be of wrought iron) after deducting for the bolt-holes

at Y should be equal to the tension in the tie-beam divided by 12,500 lbs., and the thickness of the plate *should never be less than* $\frac{3}{8}''$.

In this case we would require a net sectional area $=\frac{27,350}{12,500}=$ 2.2 sq. ins., but as the plate must be 10″ wide and $\frac{3}{8}''$ thick it will give a net area considerably in excess of this. The height H for the toe of the rafter should be equal to the tension in the tie-beam divided by the breadth of the rafter multiplied by 1,000 for white pine, 1,200 for spruce, 1,350 for oak and Oregon pine, and 1,500 for long-leaf yellow pine.

In this case the breadth of the rafter is 10 ins. and the wood is white pine; therefore H should equal $\frac{27,350}{10\times 1,000}=2\frac{3}{4}$ ins. The distance X should be sufficient to resist the tendency of the plate to shear off the top of the tie-beam, and is found by dividing the tension in the tie-beam by the breadth of the beam multiplied by the resistance to longitudinal shearing, given on p. 361, increased by 20 per cent. on account of the additional resistance to shearing caused by the vertical pressure of the strut. The answer will be in inches.

In this case X should equal $\frac{27,350}{10\times 96}=28\frac{1}{2}$ ins. In the drawing $X=30$ ins.

Bolts.—At least two bolts are always required for this joint, one at Z and one at Y, and when the breadth of the tie-plate exceeds 6 ins., there should be two bolts at Y. The bolt at Z need not exceed 1″ when the tension in the tie-beam is less than 50,000.

The diameter of the bolts at Y should be in proportion to the thickness of the plate, and the bolts should always be placed against the lug on the plate. When a joint like Fig. 9 is under pressure there is a decided tendency for the lug on the plate to spring up out of the wood, and in an actual test the spring in the plate was sufficient to break the head off of a 1″ bolt. The bolt, however, was placed about 2 ins. back from the notch. The author knows of no way by which the stress in the bolts at Y can be computed. In his judgment, however, two $\frac{3}{4}''$ bolts will be sufficient for a $\frac{3}{8}''$ plate, two 1″ bolts for a $\frac{1}{2}''$ plate, and two $1\frac{1}{4}''$ bolts for a $\frac{5}{8}''$ plate 10 ins. wide. If the plate is 12 ins. wide three bolts should be used.

Joint 2 of Fig. 1.—Where a brace abuts against a rafter, as in this joint, the end of the brace should be notched into the rafter sufficient to give it a good "hold." In this case a notch of $\frac{7}{8}''$ will be sufficient. To support the purlin, a 3″ plank may be bolted to rafter and brace, as in Fig. 10, or the rafter may be hung in duplex hangers let into the rafter. For purlins larger than $8'' \times 10''$ the duplex hangers are to be preferred.

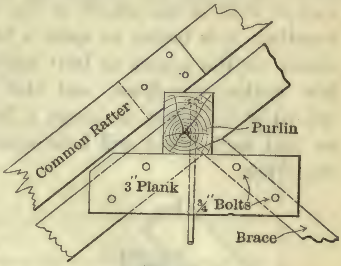


Fig. 10
Detail of Joint 2 of Fig. 1.

In the truss shown by Fig. 1, there is no rod at joint 2, but as there very often is a rod at this joint, one is shown in Fig. 10. If the rod does not exceed $1\frac{1}{4}''$ in diameter, a round hole may be bored in the top of the rafter to form a seat for a cast-iron washer.

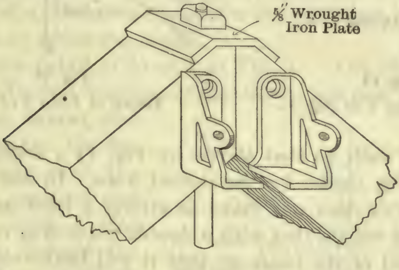


Fig. 11
Joint at Apex of King-rod Truss.

Fig. 11 shows the joint at the top of a king-rod truss, with a duplex hanger for supporting the purlin. For heavy trusses a cast-iron cap-plate such as is shown by Fig. 12 is preferable to the bent plate, unless the latter is made very heavy and lag-screwed to the rafters. The rafters should butt square against each other.

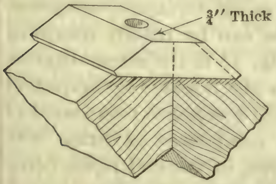


Fig. 12

Joint 3 of Fig. 1.—This should be made as shown by Fig. 13. In place of the cast washer a bent plate of wrought iron is often used.

Fig. 14 shows how joint 2 of Fig. 3 should be made, this detail applying also to any of the upper joints of a Howe truss, except that at the centre of the truss, where two braces come together, it is better to spike a block to the bottom of the top chord for the braces to butt against, as in Fig. 3, as this does not weaken the truss, and also where counter-braces are required it is better to insert a hard-wood block as in Fig. 16, so that each brace will bear against the chord independent of the other.

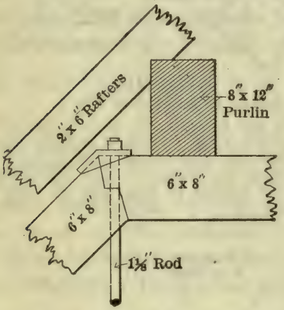


Fig. 13

Detail of Joint 3 of Fig. 1.

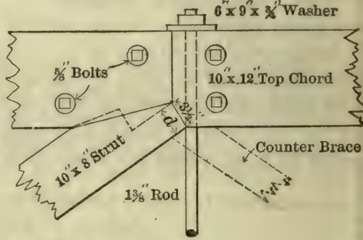


Fig. 14

Detail of Joint 2 of Fig. 3.

For joints such as that shown by Fig. 14 a double notch is often made, as shown by the dotted lines. In the opinion of the author this does not make as strong a joint as the single notch, for the reason that with a double notch it is very difficult to fit the end of the brace so that it will bear evenly in both notches, while with a single notch the full bearing must necessarily be brought on the toe.

As it is important not to cut into the chords of a Howe truss for the braces more than is really necessary, *the depth of the notch should always be proportioned to the horizontal component of the stress in the brace.* The latter can be readily measured from the stress diagram. Thus, Fig. 15 is the stress diagram of the truss shown by Fig. 3, 0–2 is the horizontal component of the stress in the end brace, 2–4 the horizontal component of brace 3–4, and 4–6 the horizontal component of brace 5–6, the horizontal component of the stress in the end brace being always equal to the tension in the tie-beam of the end panel. To avoid splintering or crushing of the wood, the depth of the

notch d (Fig. 14) multiplied by the breadth of the strut should be equal to the horizontal component in pounds divided by the values given on p. 1054 for finding the height H. For

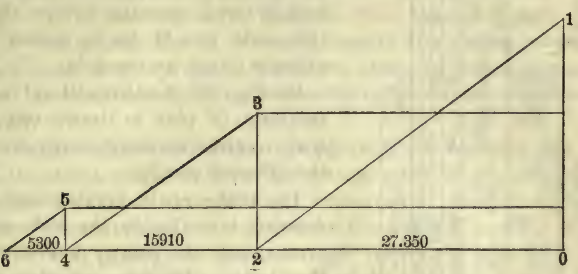


Fig. 15

the truss shown by Fig. 3, this rule would require a depth d for the outer brace of $2\frac{3}{4}''$, $1\frac{5}{8}''$ for the next brace, and $\frac{3}{4}''$ for the inner brace. A depth of $\frac{7}{8}$ inch, however, is about the minimum that should be made. As the stress is the same at both ends of a brace, the notch in the tie-beam should be of same depth as is the top chord.

Fig. 16 is a detail of joint 7 of Fig. 1. The block between the braces affords a bearing for the stirrup and also a good bearing for the braces, and does not weaken the tie-beam as much as if the block were omitted and the braces notched into the tie-beam beyond the stirrup. When a block is inserted between the ends of two braces, and especially when the braces are not subject to the same stress, the block should be notched into the tie-beam or chord about 1 in., otherwise the brace having the greater stress might push the other brace along. In the case of the two centre struts of Fig. 3 the block between their upper ends need only be spiked to the top chord, because the two braces would nearly always have the stame stress and three or four good spikes would be fully capable of resisting any difference in the stresses that might arise through unequal loading of the truss.

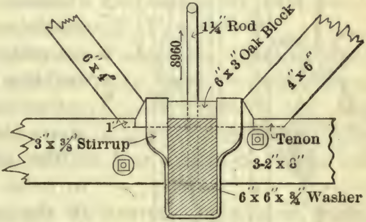


Fig. 16
Joint 7 of Fig. 1.

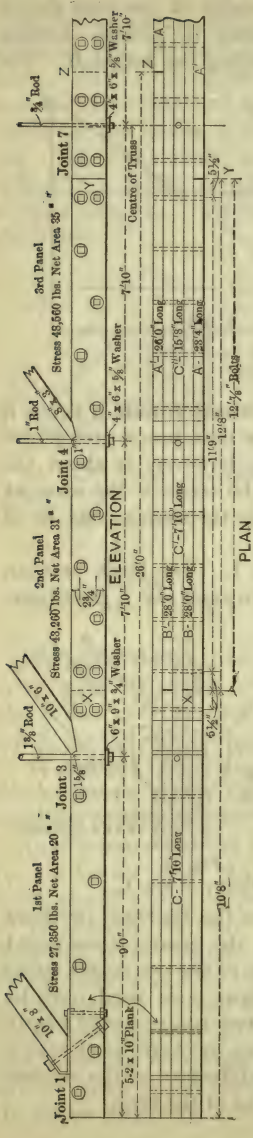


Fig. 17.—Detail of Tie-beam, Fig. 3.

Detail of Tie-beam, Fig. 3.—Fig. 17 shows a little more than one half of the tie-beam as it should be laid out in practice except that the scale should be increased to at least ½ inch to the foot.

In Fig. 17 the breadth of the tie-beam in plan is drawn out of scale in order to more clearly show the different planks.

The first step in making such a drawing is to locate the rods and braces, with the proper notches for the latter. It then remains to show the splices. In this truss the tie-beam is to be built up of 2″ × 10″ planks five layers in thickness, and the problem is how to break joints, and how many bolts are required to give the necessary tensile strength to the chord. The correct method of building up such a tie-beam is fully explained under Case 1, p. 385; therefore we will consider this example as briefly as possible. For convenience the tensile stress in the tie-beam for each panel and the net sectional area required is given above the beam, Fig. 17.

Two planks will be amply strong to resist the tensile stress even in in the central panel. The centre layer of the beam we will consider merely as filling. The problem is to bolt the other four layers together so as to form a continuous tie having the necessary tensile strength. The two outer layers we will make of two planks each—two planks *A′* 26′ 0″ long, and two *A* 23′ 4″ long (the beam being 49′ 6″ long). This will bring the joints in these

layers at Y and Z. In the next layers we will use planks 28′ long in the centre of the beam, with planks 10′ 8″ long at each end, bringing the joints in the second and fourth layers at X and the same distance from the other end.

As the planks B and B' reach beyond the centre panels on each side, they will carry the entire stress in those panels, so that we need only figure on transmitting the stress in the second panel.

We can safely assume that the plank B will transmit one half of the stress, or 21,630 lbs. This stress must be transmitted to A by bolts having a combined resistance of this amount and these bolts must be located between the joints X and Y. From Table VII, p. 383, we find that the resistance of a $\frac{7}{8}$-in. bolt in white pine is 880 lbs. per inch of length, hence in a 2-in. plank it is 1,760 lbs. $21{,}630 \div 1{,}760 = 12+$.

As we will have four bolts between Y and Z, twelve bolts will be ample between X and Y. Two of these bolts should be placed $5\frac{1}{2}''$ from Y and two the same distance from X (see last column of Table VII, p. 383), leaving eight to be spaced between, which will make the distance c. to c. $15\frac{3}{4}$ ins. As the planks A and A' extend to the end of the tie-beam, it is only necessary to use enough bolts from X to the end to hold the planks well together; $\frac{3}{4}''$ bolts, 2 ft. c. to c., will be ample for this purpose. Wherever an end joint comes in a tie-beam, two bolts should be placed each side of the joint as at X, Y, and Z.

As the centre layer will offer some assistance in transmitting the stress, the tie-beam will probably have some excess of strength even with a good factor of safety; but, on the other hand, some of the bolts may not fit perfectly and the planks may not be full 2 ins. thick, so that it is well to be on the safe side.

Wall Joint of Scissors Trusses.—In scissors trusses the joint over the wall formed by the rafter and tie-beam should always be carefully proportioned to the stress in the tie, otherwise the joint is liable to open and allow the wall to be pushed out. Much greater strength is required in this joint than in the wall joint of a king-rod truss of the same span, because the stresses in a scissors truss are usually at least twice and sometimes three or four times as great as in a truss with a horizontal tie-beam. For a scissors truss built of planks as in Fig. 2 a $\frac{3}{4}''$ bolt through the centre of each joint, with as

many spikes as can be driven, will ordinarily give sufficient strength.

For trusses like those shown by Figs. 27–30 of Chapter XXV the author has found that the best method of making the wall joint, unless the roof is quite flat, is that shown by Fig. 18, which is the detail of an actual joint used by the author where the stress in the tie-beam was 25,000 lbs.

It should be noticed that the wrought-iron strap is secured to the tie by lag-screws instead of bolts. The author has found

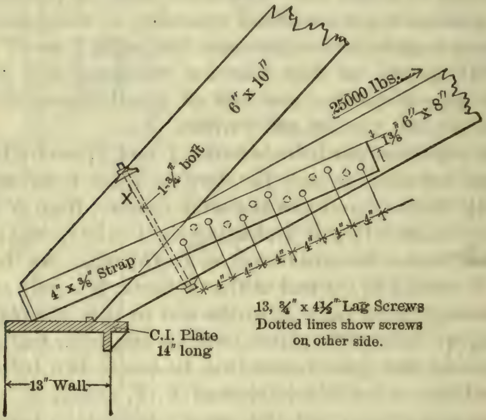


Fig. 18
Wall Joint of Scissors Truss.

that it is practically impossible to *bolt* a strap to each side of a beam so as to get a good bearing for all of the bolts, owing to the difficulty in boring the holes straight, and if the holes are bored a little large some bolts may bear on the wood and some may not.

With lag-screws each screw is bound to get a good bearing in the wood. The holes in the two sides of the strap must, of course, be staggered, so that they will not come opposite each other.

The net sectional area of the strap should at least be equal to the stress in the tie-beam divided by 20,000 lbs.

The number of lag-screws (for both sides) is found by dividing the stress in the tie-beam by the resistance of one screw. For the safe resistance of lag-screws used in this way the author recommends the values given in Table I.

In the joint shown by Fig. 18 the stress in the tie-beam is 25,000 lbs. and the wood is Oregon pine.

The above rules therefore require a sectional area in the strap $=\frac{25,000}{20,000}=1\frac{1}{4}$ sq. ins. and twelve $\frac{3}{4}''$ lag-screws.

TABLE I.—SAFE RESISTANCE OF LAG-SCREWS WHEN USED AS IN FIG. 18.

Size of Screw.		Safe Resistance in Pounds.				Minimum Thickness of Strap.
		Oak.	White Pine.	Oregon Pine.	Georgia Pine.	
Dia.	Length.					
$\frac{3}{8}$ ×	$3\frac{1}{2}$	800	600	700	800	$\frac{1}{4}$
$\frac{1}{2}$ ×	4	1400	1000	1100	1200	$\frac{1}{4}$
$\frac{5}{8}$ ×	4	2000	1500	1650	1800	$\frac{5}{16}$
$\frac{3}{4}$ ×	$4\frac{1}{2}$	2500	1800	2100	2400	$\frac{5}{16}$
$\frac{7}{8}$ ×	5	3000	2400	2800	3000	$\frac{3}{8}$

With a thickness of $\frac{3}{8}$ in., the width of the strap necessary to give a sectional area of 1.25 sq. ins. $=\frac{1.25}{.375}=3\frac{1}{3}$ ins. To this should be added the diameter of one lag-screw to obtain

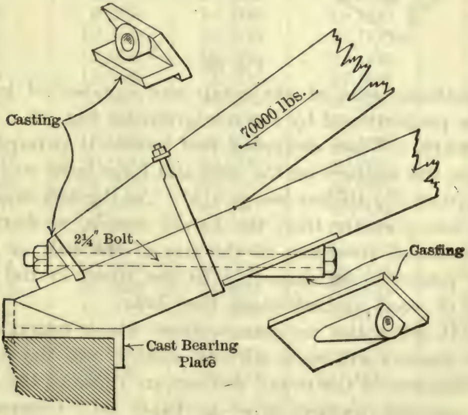


Fig. 19
Wall Joint of Scissors Truss.

the working width, $3\frac{1}{3}+\frac{3}{4}=4\frac{1}{12}$ ins. The strap used was $4''\times\frac{3}{8}''$, as some additional strength was obtained by the bolt at X,

which it is necessary to insert to hold the timbers together while the truss is being raised into position, and also to bring them tightly together before fitting the strap.

Fig. 19 shows another method of making this joint which may be used with advantage when the inclination of the rafter is less than 45 degrees. This joint has the advantage that if the truss is erected one piece at a time the tie-beams may be put up first and a seat is provided to receive the rafters. The strap prevents the end of the rafter from springing up. The diameter of the bolt should be proportioned to the horizontal component of the stress in the rafter using the value for strength given in Table IX, p. 384. Fig. 20 shows a good form of joint to use at joint 5 of Fig. 30, p. 903, when it is desired to substitute a wooden tie for the rods shown in Fig. 30.

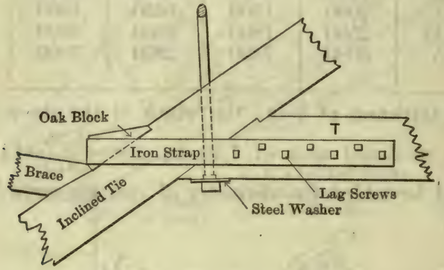


Fig. 20

The sectional area of the strap and number of lag-screws should be proportioned by the rule given for Fig. 18.

Washers.—When designing roof trusses it is important to proportion the washers on the rods and large bolts so that they will not crush the timber (see p. 414). As the soft woods crush under a less pressure than the harder woods, washers cannot be proportioned according to the size of the bolt or rod, but must be proportioned according to the stress in the rod and the kind of wood against which they bear.

Table II gives the maximum stress which round and rectangular washers will resist without sinking into the wood.

The diameters of the round washers are those of the standard sizes of cast-iron washers given in Table III. Comparing the values given in Table II for round washers with the strength of the rods for which they are intended, it will be seen that the bearing resistance of the washers on white pine and spruce is

only about one half the working strength of the rod, consequently for white pine and Oregon pine the standard size of washers is not large enough for the strength of the rod.

TABLE II.—SAFE BEARING RESISTANCE OF WASHERS IN POUNDS.

ROUND WASHERS.

Diameter.	White Pine and Spruce.	Oregon Pine.	Georgia Pine.	Oak.
$2\frac{5}{8}$	1,350	2,160	2,700	3,240
3	1,760	2,820	3,520	4,230
$3\frac{1}{4}$	2,070	3,300	4,140	4,970
$3\frac{3}{4}$	2,760	4,400	5,520	6,620
4	3,140	5,020	6,280	7,530
$4\frac{3}{4}$	4,430	7,080	8,860	10,630
6	7,060	11,400	14,100	16,960
$6\frac{1}{4}$	7,660	12,260	15,300	18,400
$7\frac{1}{4}$	10,300	16,500	20,600	24,700
$8\frac{1}{4}$	12,900	20,700	25,800	31,100
$9\frac{1}{4}$	16,250	26,000	32,500	39,000
$10\frac{1}{4}$	20,000	32,000	40,000	48,000

RECTANGULAR WASHERS.

Size.	White Pine and Spruce.	Oregon Pine.	Georgia Pine.	Oak.
4×6	6,000	9,600	12,000	14,400
4×8	8,000	12,800	16,000	19,200
6×6	9,000	14,400	18,000	21,600
6×7	10,500	16,800	21,000	25,200
6×8	12,000	19,200	24,000	28,800
6×9	13,500	21,600	27,000	32,400
6×10	15,000	24,000	30,000	36,000
8×8	16,000	25,600	32,000	38,400
8×9	18,000	28,800	36,000	43,200
8×10	20,000	32,000	40,000	48,000
8×12	24,000	38,400	48,000	57,600
10×10	25,000	40,000	50,000	60,000
10×11	27,500	44,000	55,000	66,000
10×12	30,000	48,000	60,000	72,000
10×14	35,000	56,000	70,000	84,000
12×12	36,000	57,600	72,000	86,400
12×14	42,000	67,200	84,000	100,800
12×16	48,000	76,800	96,000	115,200
14×14	49,000	78,400	98,000	117,600
14×16	56,000	89,600	112,000	134,400

As a rule for the rods of wooden trusses it is best to use rectangular washers cut from steel plates, cutting the washers to the required size. It is, of course, not really dangerous to

use smaller washers than would be required by Table II, as a little crushing of the timber will not endanger the safety of the truss, but it is best to keep within the limits of Table II when practicable.

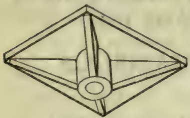
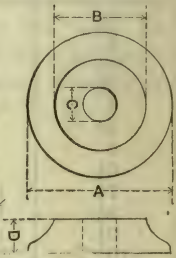


Fig. 21

Very large washers should be made of cast iron with brackets, as in Fig. 21. The rod in Fig. 16 has a stress of 8,960 lbs., and as the wood is white pine, we see from Table II that a $6'' \times 6''$ washer will be required.

TABLE III.—PROPORTIONS OF STANDARD CAST WASHERS.



For sizes not given below.

Diameter of bolt $= d$.

$A = 4d + \frac{1}{4}''$. $C = 1d + \frac{1}{8}''$.

$B = 2d + \frac{1}{4}''$. $D = 1d$.

All dimensions in inches.

Standard Cast Washer.

Diameter of Bolt $= d$.	*A.*	*B.*	*C.*	*D.*	Weight in Pounds.
$\frac{1}{2}$	$2\frac{5}{8}$	$1\frac{3}{4}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{1}{2}$
$\frac{5}{8}$	3	$1\frac{7}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{3}{4}$
$\frac{3}{4}$	$3\frac{1}{4}$	$2\frac{1}{8}$	$\frac{13}{16}$	$\frac{7}{8}$	$1\frac{1}{4}$
$\frac{7}{8}$	$3\frac{3}{4}$	$2\frac{1}{2}$	$\frac{15}{16}$	$\frac{7}{8}$	$1\frac{1}{2}$
1	4	$2\frac{3}{4}$	$1\frac{1}{16}$	$1\frac{1}{8}$	$2\frac{1}{2}$
$1\frac{1}{8}$	$4\frac{3}{4}$	$2\frac{3}{4}$	$1\frac{3}{16}$	$1\frac{1}{8}$	3
$1\frac{1}{4}$	6	3	$1\frac{5}{16}$	$1\frac{3}{8}$	$5\frac{3}{4}$
$1\frac{1}{2}$	$6\frac{1}{4}$	$3\frac{1}{4}$	$1\frac{5}{8}$	$1\frac{1}{2}$	6
$1\frac{3}{4}$	$7\frac{1}{4}$	$3\frac{3}{4}$	$1\frac{7}{8}$	$1\frac{3}{4}$	$9\frac{1}{2}$
2	$8\frac{1}{4}$	$4\frac{1}{4}$	$2\frac{1}{8}$	2	$17\frac{1}{4}$
$2\frac{1}{4}$	$9\frac{1}{4}$	$4\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{1}{4}$	20
$2\frac{1}{2}$	$10\frac{1}{4}$	$5\frac{1}{4}$	$2\frac{5}{8}$	$2\frac{1}{2}$	$27\frac{1}{4}$
$2\frac{3}{4}$	$11\frac{1}{4}$	$5\frac{3}{4}$	$2\frac{7}{8}$	$2\frac{3}{4}$	36
3	$12\frac{1}{4}$	$6\frac{1}{4}$	$3\frac{1}{8}$	3	46

Riveted Joints of Steel Trusses.

Trusses with riveted joints are invariably made with angle-bars for the web members and generally for the chords, although the latter are sometimes made of a pair of channels

or of two angles and a web-plate. The members are connected at the joints by means of *gusset-plates*, to which all of the members are riveted. Typical examples of riveted joints in roof trusses are shown by Figs. 24 to 34. When the rafter or chord has a web-plate, as in Fig. 26, the web members are riveted to this plate and a gusset-plate is not required except at the end joint and apex.

In order that there shall be no twisting, it is necessary that the principal members of the truss be double so that the gusset-plate may be riveted between them. Where single angles are used for web members and two such members come at one joint they should be riveted to opposite sides of the gusset-plates as in Fig. 31. The thickness of the gusset-plates, as a rule, should

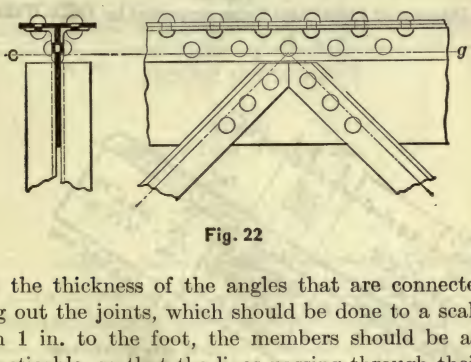


Fig. 22

be twice the thickness of the angles that are connected to it. In laying out the joints, which should be done to a scale of not less than 1 in. to the foot, the members should be arranged, when practicable, so that the lines passing through their centre of gravity will coincide with the lines of the truss diagram, and thus meet at a single point, as in Fig. 22. This is not always practicable, but the principle should be followed as closely as possible. For small angles the rivet lines of the members may be considered as passing through the centre of gravity of the section without serious error.

The number of rivets required for each member must be determined according to the stress in the members, the resistance of the rivets being considered both for shearing and bearing.

The method of determining the number of rivets in a joint is explained on pp. 363–370, but to show more clearly the application to truss joints we will illustrate by one example.

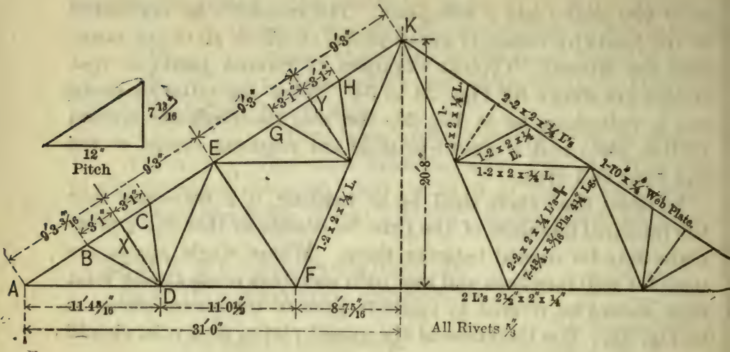


Fig. 23
Diagram of Light Steel Truss supported by Brick Walls.

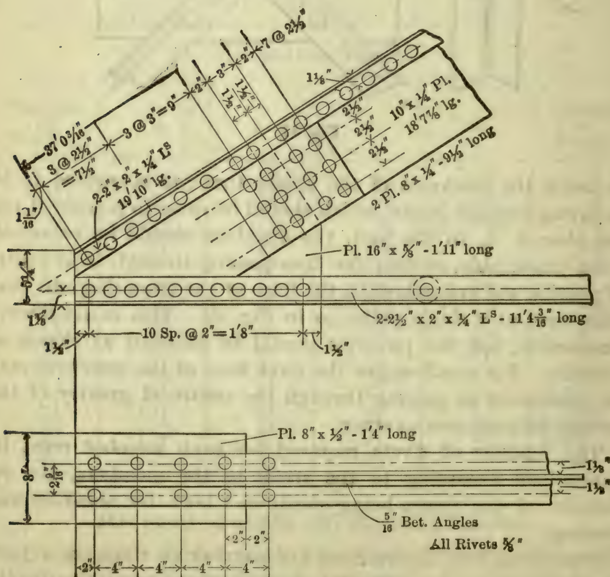


Fig. 24
Detail of Joint *A* of Fig. 23. All Rivets ⅝ inch.

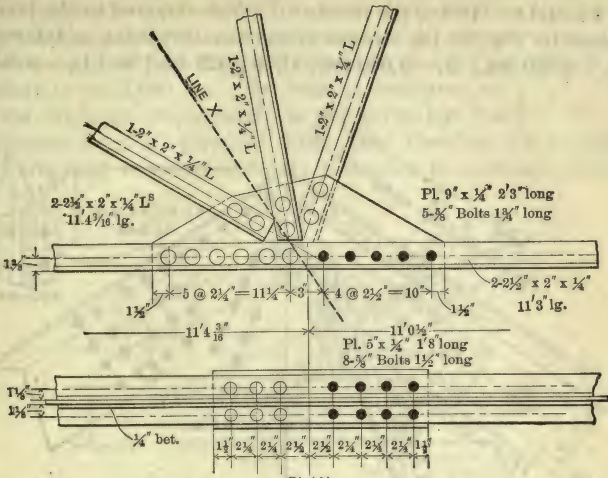


Fig. 25

Detail of Joint *D* of Fig. 23.

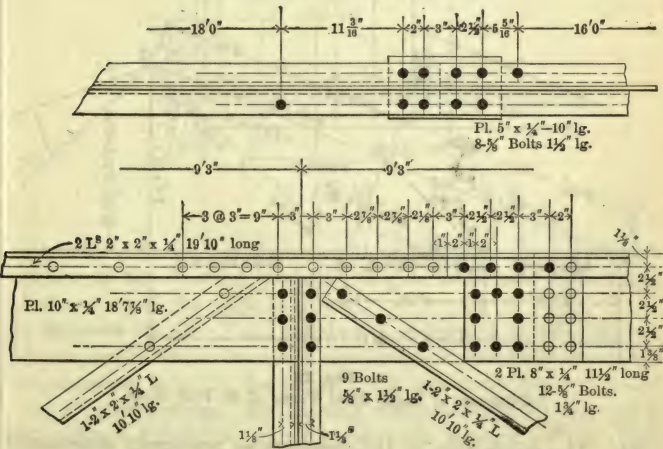


Fig. 26

Detail of Joint *E* of Fig. 23. All rivets ⅝ inch.

EXAMPLE.—To find the number of rivets required in the joint shown by Fig. 32, the stresses in the members being as follows: *A*, −6,250 lbs.; *B*, −3,050 lbs.; *C*, +2,325 lbs.; and *D*, −9,300

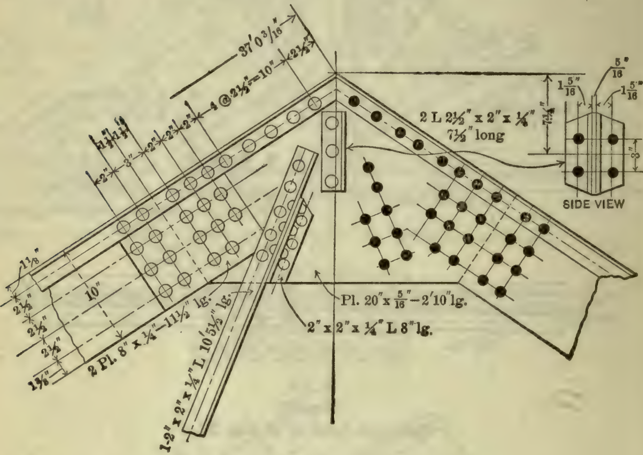


Fig. 27
Detail of Joint *K* of Fig. 23.

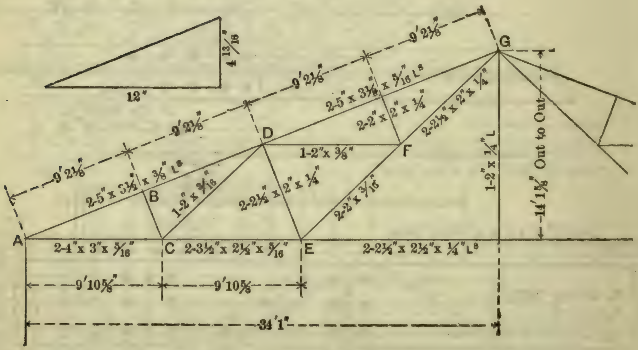


Fig. 28
Diagram of Light Steel Truss of 68 Feet Span.

lbs. The dimensions of the members as previously determined are given in the figure.

We will use a gusset-plate $\frac{3}{8}''$ thick.

Number of Rivets Required for B.—As there is but one angle the rivets will be in single shear, and as the leg of the angle is only 2″ wide we must use ⅝″ rivets. From the table on p. 372 we find the resistance of a ⅝″ rivet to single shear to be 3,060 lbs. The bearing resistance on a $\frac{3}{16}$″ plate (the thickness of the angle) is not given, but it is ¾ths the resistance for a ¼″ plate, or 2,100 lbs.; therefore the strength of the rivet is governed by its resistance to bearing. As the

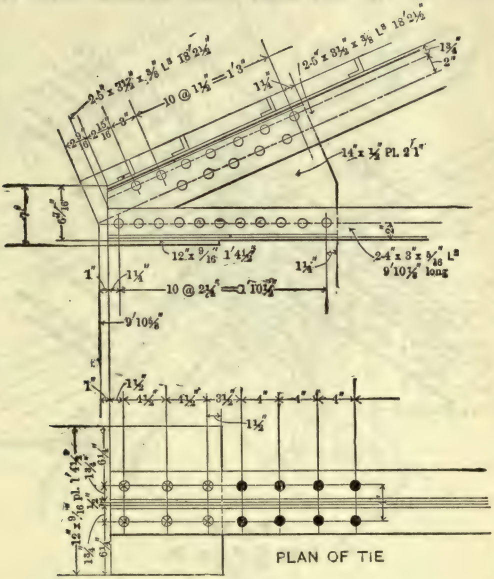


Fig. 29
Joint *A* of Fig. 28.

stress is 3,050 lbs. it will require 1½ or 2 rivets. Four rivets are shown in the drawing, probably to give additional stiffness, as but one leg of the angle is riveted.

Rivets in C.—This member is composed of two angles, consequently the rivets are in double shear, and their resistance to shearing is 6,120 lbs. each. The minimum bearing is on the ⅜″ gusset-plate. The bearing resistance of a ⅝″ rivet on a ⅜″ plate is 4,210 lbs., which governs the strength of the rivet. As the stress is only 2,325 lbs. only one rivet would, theoretically,

be required to resist the stress, but two rivets is the least that should ever be used in the end of a truss member, no matter how small the stress.

Rivets in D.—This member is also double, and as the combined thickness of the angles is greater than the thickness of the gusset-plates the strength of the rivets will be governed by the resistance to bearing on a $\frac{3}{8}''$ plate, or 4,210 lbs. As the stress is 9,300 lbs., it will require three rivets to resist it and two rivets for *A*. For such small stresses more rivets are

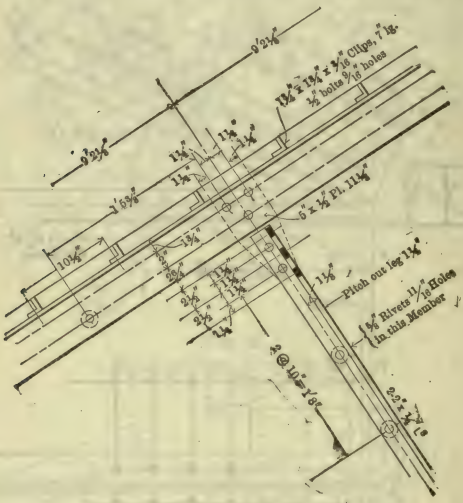


Fig. 30
Joint *B* of Fig. 28.

generally used than are theoretically required, but when the stresses are large, only as many rivets as are theoretically required are generally used.

The above example illustrates the process to be pursued in determining the number of rivets in any joint.

For Angles in Tension, both legs should be connected by rivets, as in Fig. 33, unless the sectional area of the angle is very much greater than would theoretically be required, as is the case with the brace *B* in the last example.

Cooper, in his "Specifications for Iron and Steel Bridges," requires that "Angles subject to direct tension must be con-

nected by both legs or the section of one leg only will be considered as effective."

When two rows of rivets are used in a tension member, the rivets should be staggered as in Fig. 34, so that any cross-section will be weakened by only one rivet-hole.

Stay-rivets.—Struts composed of a pair of angles should be riveted together, with a washer between the angles, about

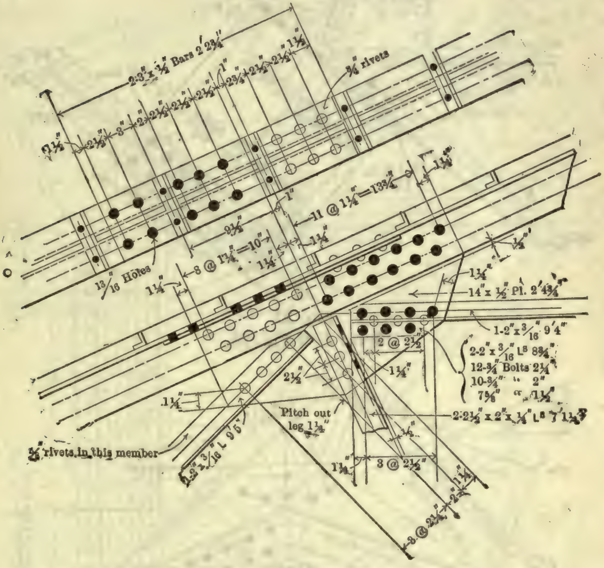


Fig. 31
Joint *D* of Fig. 28.

every eighteen inches for 2″ angles, 24″ for $2\frac{1}{2}''$ angles, and 30″ for $3\frac{1}{2}''$ angles, and from 30″ to 3 ft. for tension members. Stay-rivets are shown in Figs. 30 and 34.

For locating the rivet lines on angles, or the "pitch," as it is called in bridge-shops, the distances given in the table on p. 551 should be used.

Figs. 24–27 show the details of several of the joints in the truss shown by the diagram, Fig. 23, and Figs. 29–33 several of the joints of the truss shown in Fig. 28. The engravings were made from the actual working drawings prepared by the

Berlin Iron Bridge Co., and are very good examples of riveted joints in roof-trusses.

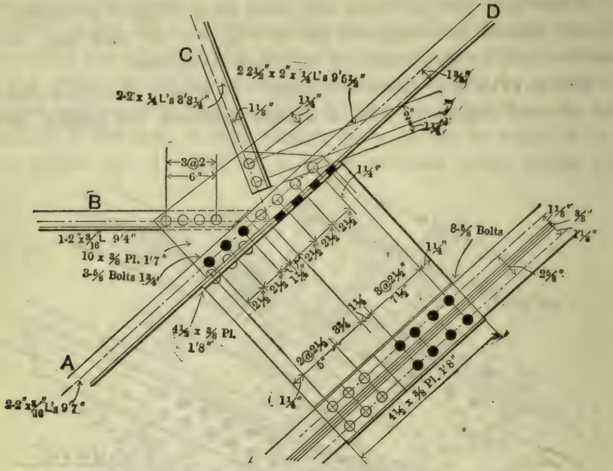


Fig. 32
Joint *F* of Fig. 28.

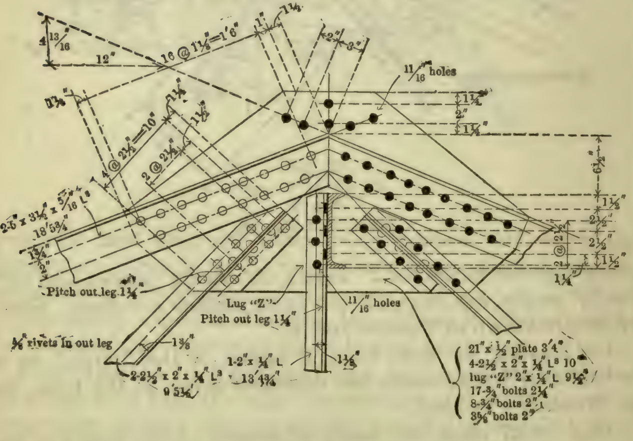


Fig 33.
Joint *G* of Fig. 28.

The solid black circles indicate holes for bolts to be put in

place in the field, the truss being shipped in four parts and assembled at the building.

Fig. 34 was engraved from the working-drawing made by the

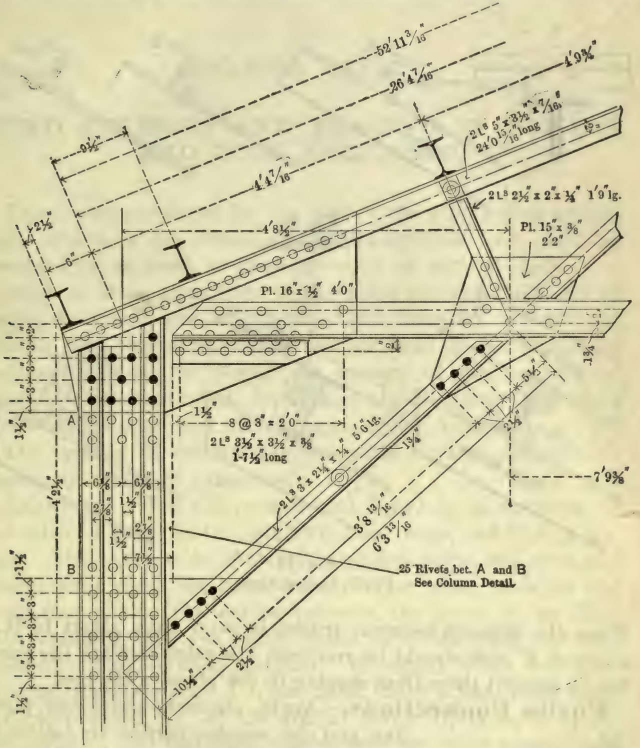


Fig. 34

New Jersey Steel and Iron Co. of Trenton, N. J., and represents joint 1 of Fig. 60, p. 922.

Purlins and Purlin Connections.

Where the roofing is supported directly on the purlins, as is generally the case in light steel roofs, the purlins and trusses are generally spaced so close together that simple shapes may be used for the former.

For spans between trusses of 8 or 10 feet, angles are commonly used, and for greater spans Z bars, channels, and I beams.

Wooden purlins are also frequently used with steel trusses.

If the purlins support wooden rafters or plank roofing, a strip of wood is bolted to the I beam or channel purlin, as shown by Figs. 36 and 37, to form nailings for the rafters or plank.

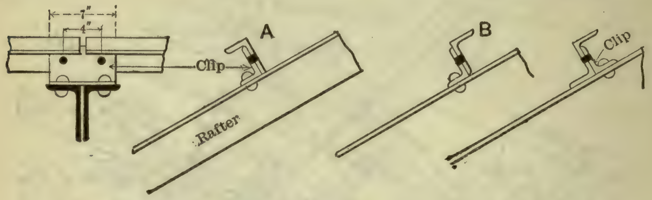


Fig. 35
Purlin-clips.

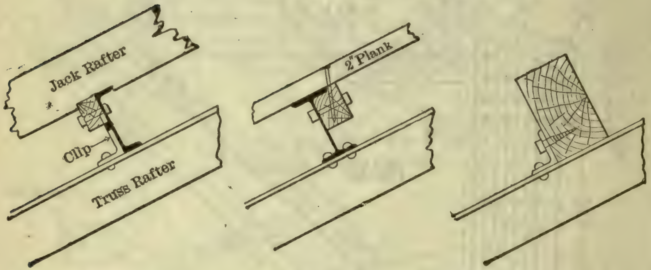


Fig. 36
Purlin Connections.

When the distance between trusses is more than about 15 ft., a line of $\frac{5}{8}''$ rods should be run from the ridge through the purlins to prevent them from sagging in the plane of the roof.

Purlin Connections.—Angle, channel, and Z-bar purlins, and also wooden purlins, are fastened to the rafters of the truss by angle-clips as in Figs. 35, 36, and 37. These clips should be riveted to the truss at the shop, the purlins being secured to the clips by bolts.

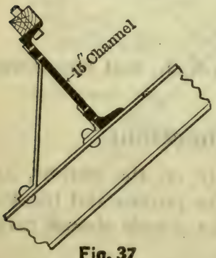


Fig. 37

I-beam purlins are usually bolted directly to the rafter by a bolt in each side of the flange. $12'' \times 15''$ I beams and channels are sometimes braced as in Fig. 37, the necessity for bracing depending largely upon the inclination of the roof.

CHAPTER XXVIII.

WIND STRESSES AND BRACING IN TOWERS AND HIGH BUILDINGS.

The stresses produced by the force of the wind acting against high structures are often of great magnitude and must be provided for as much as those produced by vertical loads.

Brick or stone structures, if the walls are well built and of proper thickness, are able to withstand these stresses without bracing, but framed structures require bracing and additional sectional area in the posts; the amount of bracing and increase in sectional area depending largely upon the height and width of base, and the character of the construction.

The method of determining the wind stresses can best be shown by means of examples.

Example 1.—We will assume that Fig. 1 is an elevation of one side of a tower 48 ft. high, 12 ft. 8 ins. wide, and 25 ft. 4 ins. long. The tower to be built with wooden posts and girts and braced with rods, the latter being in the same plane as the posts and girts. The spaces between the posts to be filled with studding, and the entire tower to be sheathed and covered with some suitable material. In regard to its resistance to wind pressure, such a frame is in effect a cantilever truss fixed at one end, i.e., at the ground (either by bolts or by its own weight), and uniformly loaded over its entire length, and the stresses may be found exactly in the same way as for a cantilever truss.

The wind pressure is considered as acting horizontally and applied at the joints the same as the loads on a truss.

In a tower, the two sides parallel with the direction of the wind are assumed to resist the stress, so that the frame shown by Fig. 1 has to resist only one half of the total pressure.

Assuming the wind to act from the left, the rods should be placed as in the figure.

The wind load at joint *o* will equal half the height of the

panel multiplied by half the width of the structure, and the product by the pressure per square foot.

For enclosed towers, the wind pressure should be taken equal to 40 lbs. per square foot of vertical elevation, at least, and if the tower is in a very exposed situation, it will be safer to assume a pressure of 50 lbs.*

Assuming a pressure of 40 lbs., we have for the loads at joints *o* and 8, of Fig. 1, $6' \times 12'\ 8'' \times 40 = 3{,}040$ lbs., or say 3,000 lbs., and at joints 2, 4, and 6 twice this amount, or 6,000 lbs. To draw the stress diagram, Fig. 1_A, we will commence with joint 1, Fig. 1 being lettered as for a truss diagram (see p. 971). The pressure exerted at *o* is transmitted by the top girt directly to joint 1, and we may therefore consider the force as acting at that joint. Then at joint 1 we have a horizontal force of 3,000 lbs., which is represented by the line *ab* drawn to a scale, and the force acts in the direction indicated. Besides this force we have the stresses in *BF* and *FA*, which we obtain by drawing a vertical line from *a*, and a line from *b*, parallel to the diagonal *B′F*, the two lines intersecting at *f*. The triangle of forces for joint 1, then, is *ab*, *bf*, and *fa*. The arrow-head on *bf* points from the joint, and on *fa*, towards the joint; hence *B′F* is in tension and *FA* in compression. (Note. It will be seen that the order of rotation is from right to left, and this order must be followed at all of the joints.)

At joint 2 we have the stress in *FB′*, which must act from the joint, next the load of 6,000 lbs., which we lay off from *b* to *c*, to the same scale as *ab* (theoretically there will be no stress in *BB′*). Then draw *cg* and *fg* to represent the stresses in *CG* and *FG* respectively. The arrow-head on *cg* points from the joint, hence *CG* is in tension. At joint 3 we have the

* In a paper read before the American Society of Civil Engineers, Mr. Julius Baier stated that the St. Louis tornado of 1896 "gave evidence that wind pressures existed at least equivalent to or greater than 20 lbs. 60 lbs., and 85 to 90 lbs. per square foot over considerable areas, and that the pressures at higher altitudes were more severe than those measured."

After a thorough study of the effects of the wind pressure during this tornado Mr. Baier recommended "that the safety and interests of the community and of the owner of the building require a recognition of a wind pressure of at least 30 lbs. per square foot against the exposed surface of a building, with an additional local provision of 50 lbs. for several stories near the top; and that this amount should be safely taken care of by some positive and definite provision in the construction of the frame."

Railway structures and steel buildings are commonly designed to resist a horizontal wind pressure of 30 lbs. per square foot.

stresses *af* and *fg*, and draw *gh* and *ha* parallel respectively to *GH* and *HA*. The stresses at joints 4 and 6 are found in the same way as at joint 2, and those at joints 5 and 7 in the same way as those at 3. At joint 8 we have the stresses *lk* and *ke*, and measure off the load of 3,000 lbs. represented by *em*.

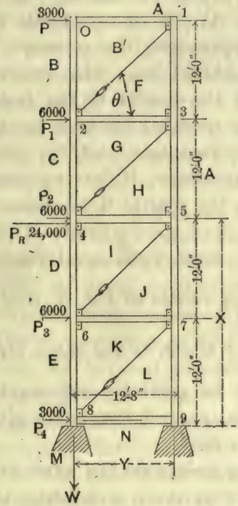


Fig. 1

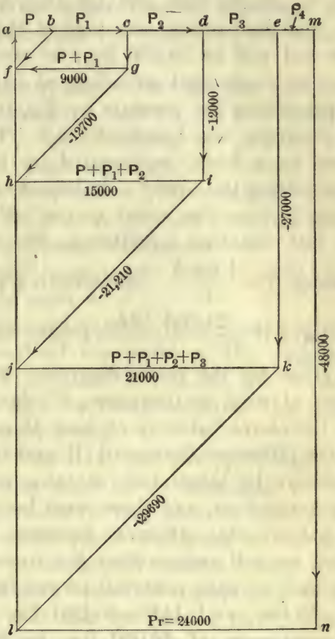


Fig. 1A

Then, if from *m* we draw a vertical line, and from *l* a horizontal line, the two intersecting at *n*, *mn* will denote the anchorage required for the post *EK*, and *ln* the stress in the girt or sill, on the assumption that the entire horizontal thrust or tendency to slide on the foundation is resisted by joint 9. In practice it is customary to fasten the frame to each of the piers, and to make the girt or sill strong enough to resist one half of the thrust.

The stress denoted by *mn* will be offset to a considerable degree, if not entirely overbalanced, by the weight of the frame

and its load. In a light frame supporting no load, the tension in the windward columns will be greater than the compression, and the columns must be spliced to resist an upward pull and be anchored to the foundation.

The uplift at joint 8 can also be obtained by taking moments about joint 9. Thus the total wind pressure acting against the windward side will be $48' \times 25'\ 4'' \times 40$ lbs. $= 48,640$ lbs. As each end resists one half of the pressure, the pressure for one end will be 24,320 lbs., or say 24,000 lbs. This pressure may be considered as acting at half the height of the frame. Representing the pressure by P_R, the moment about 9 tending to overturn the tower $= P_R \times X$. To maintain stability there must be a force, represented by the arrow, W, acting down. Considering the tower as balanced on the pier at 9, the force W tends to turn the tower to the left, and its moment $= W \times Y$. To just maintain equilibrium $W \times Y$ must just equal $P_R \times X$, whence $W = \frac{P_R \times X}{Y}$. Substituting the values of P_R, X, and Y, we have $W = \frac{24,000 \times 24}{12} = 48,000$ lbs., which is the same stress as given by the stress diagram. To be perfectly safe against gusts of wind or tornadoes, W should exceed the value given by the above value by at least 25 per cent.

The difference between $1\frac{1}{4}W$ and the weight on the post must therefore be taken care of by a rod or strap extending into the foundation, and there must be sufficient masonry provided to balance this difference between $1\frac{1}{4}W$ and the dead weight. Thus we will assume that our tower weighs 80,000 lbs., then the load on each post will be one fourth of this, or 20,000 lbs., $W = 48,000$, and $1\frac{1}{4}W = 60,000$ lbs. Therefore it will require an anchorage of 40,000 lbs. to secure absolute safety. As the weight of stone masonry may be taken at 140 lbs. per cubic foot, it will require $\frac{40,000}{140} = 286$ cu. ft. to hold the tower down.

To the compression in the leeward columns due to wind pressure must be added the compression due to the vertical loads. Thus if the post 7–9 has to support a dead load of 20,000 lbs., it should be made large enough to sustain $20,000 + 48,000$, or 68,000 lbs. In computing the size of struts or ties to resist wind pressure, however, a greater unit stress or a smaller factor of safety is generally used than for other loads.

In Fig. 1 only one diagonal is shown for each panel, and if we could be sure that the wind would always blow from the left, that is all that would be required.

As the wind may blow from any direction, however, it is necessary to insert diagonals in both directions, and in each of the four sides of the tower, and if it is necessary to provide any anchorage at all, all of the posts must be anchored, and each post must be proportioned for the maximum net tension and maximum compression.

Analysis of Stress Diagram.—The stresses given on Fig. 1A were obtained by scaling the lines by the scale used in laying off the pressures P, P_1, etc. Studying the diagram and stresses, it will be seen that the compression (or shear, as it is commonly called) in 2–3$=P+P_1$. The shear in 4–5$=P+P_1+P_2$, and so on. *Or the shear in any girt equals the panel load at that point plus all of the panel loads above.*

Also, the stress in the diagonals equals the shear in strut above multiplied by the secant of the angle θ.

The compression in the leeward column from 1 to 3 equals af, or the vertical component of the stress in the diagonal. The compression from 3 to 5 equals $hf+fa$, or the vertical component of the diagonal $hg+$the compression in the panel above, and this is true for every panel.

Now the line $af=P$ times tangent θ, and $fh=fg\times$tangent θ. If we denote the vertical component of the diagonals by the term *increment*, then the *increment for any panel is equal to the shear in the strut at the top of the panel multiplied by the tangent* θ, and the *compression in the leeward column in any panel equals the increment for that panel plus the compression in the panel above.* It can also be seen that the tension in CG equals the compression in AF, and the same relation is true for the panels below, or the *tension in the windward column in any panel is equal to the compression in the leeward column in the panel above.*

These facts enable one to readily compute the stresses in a simple frame like Fig. 1, without drawing the stress diagram, and the wind stresses in the framework of high buildings are usually computed directly by means of the above propositions, as is shown on p. 1088.

If the tower is battering, however, the above propositions do not hold true, and the stresses can be most easily obtained by drawing a stress diagram.

Example 2.—To show the application of the graphic method of finding the wind stresses when the columns are inclined, we will determine the stresses for the water-tower shown by Fig. 2. We will assume that the tower is square in plan and open on the sides. The tank is supposed to be circular and to weigh, when two thirds full of water, about 80,000 lbs The area of the tank exposed to the wind is its diameter multiplied by the average height, or $12 \times 13 = 156$ ft. For the pressure per square foot of vertical surface we will assume 60 lbs. to provide against any possible wind. This would make the pressure on 156 sq. ft. 9,360 lbs. For a circular tower or chimney, however, it is customary to consider the pressure as only two thirds what it would be on a flat surface, which would make the pressure on the tank 6,240 lbs., or 3,120 lbs. on each side. For the pressure on the tower 20 lbs. per square foot of *vertical* elevation should be sufficient (considering that it is an open frame). This will give the panel loads to be resisted by each side, as indicated in the figure, the load at the top of the frame including both the pressure on the tank and the pressure on the upper half of the frame.

The stress diagram (Fig. 2A) is drawn in exactly the same way as Fig. 1A, commencing by drawing the line *ab*, equal to the pressure at joint 1, and from *a* and *b* lines parallel respectively to *AF* and *B′F*. At joint 2 we have *fb*, and draw *bc* equal to the pressure at that joint. Then from *c* draw a line parallel to *CG*, and from *f* (the point of beginning) a horizontal line, the two lines intersecting at *g*. At joint 3 the polygon of forces is *af*, *fg*, *gh*, and *ha*.

The stresses at joints 4 and 6 are drawn in the same way as those at joint 2, and the stresses at joints 5 and 7 in the same way as those at joint 3.

At joint 8 we have *lk* and *ke*, and draw *em* = to the pressure; from *m* draw a vertical line, because the weight of the anchorage will be vertical, and from *l* a horizontal line. Then *mn* will represent the upward pull on the windward piers due to wind pressure, and *ln* the compression on the bottom girt, assuming that the entire stress is transmitted to joint 9.

As all of the posts will be fixed, however, the girt need be proportioned to resist only one half of the stress shown by *ln*. The stresses given in Fig. 2A were obtained by scaling the lines. It will be seen that all of the stresses are considerably less than they would be if the tower were of the same width

at the base that it is at the top, or, in other words, the resistance of the tower is materially increased by inclining the posts. In proportioning the girts and diagonals, only the stress due to wind pressure need be taken into account, as the vertical load would produce no stress in the diagonals, and practical requirements will cause the girts to be made greater than would

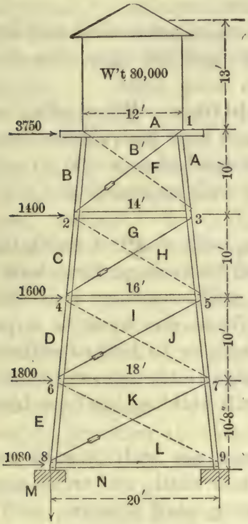


Fig. 2

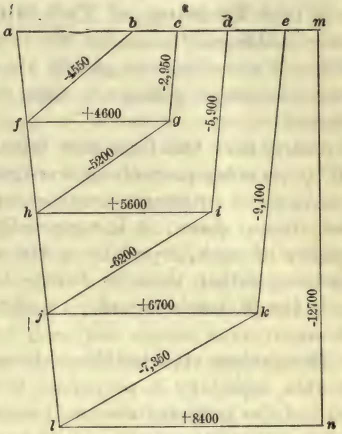


Fig. 2A

theoretically be necessary. The posts, however, must be proportioned for both the dead load and wind stress.

Allowing 8,000 lbs. for the weight of the frame, the dead load on each post will be 22,000 lbs., and as the wind stress on the leeward posts between joints 7 and 9 is 12,800 lbs., all four of the posts should be proportioned to support 34,800 lbs. compression.

The tension in the windward posts is only 9,100 lbs., and as this is greatly exceeded by the dead load, it need not be taken into account. The uplift on the windward piers is 12,700 lbs., but as the dead load is 22,000 lbs, no anchorage will be required, although it would be well to bolt the posts to the piers by $\frac{3}{4}''$ or $\frac{7}{8}''$ bolts 3 or 4 ft. long.

The diagonals should, of course, be run in both directions,

in all four sides of the tower, and should be provided with turnbuckles, so that the rods may be well tightened after the frame is erected.

In these two examples we have considered towers of only moderate height, for convenience of illustration, but the analysis is precisely the same for a frame of any number of panels or stories.

A good example of a steel water-tower is described and illustrated in the *Engineering Record* of June 20, 1903, the stress diagrams and details of construction being given.

Wind Bracing of Tall Buildings.—When office and other buildings of six to ten stories were built with solid masonry walls no attention was paid to the lateral strains due to wind pressure, except, perhaps, to make the walls and partitions a little heavier.

And as such buildings were seldom built of a less width than 50 ft., no other precautions were really necessary, for whenever buildings of ordinary construction with masonry walls have been blown down, it has generally been due more to a poor quality of work, especially in the walls, or to lack of sufficient anchors, rather than to faulty design, although occasionally, as in the St. Louis tornado, a well-built building has been blown down.

The modern steel buildings, however, are built to such great heights, especially in proportion to their width, and are so destitute of the ordinary means of resisting wind pressure, such as solid walls and partitions, that some efficient means of bracing the steel frame would seem to be a matter of necessity. As a matter of fact, few if any skeleton steel buildings are now erected without some provision for bracing the steel frame, independent of the partitions.

In some buildings these provisions consist merely in using girders built of angles and plates of good depth, in the use of riveted connections at the columns, and in breaking joints of columns at the different floor-levels, while in others heavy sway-bracing, knee or portal bracing, or both combined have been employed.

In fact, the diversity of practice as regards the wind bracing of buildings is much greater than in any other feature of construction.

Buildings which Require Bracing.—It is generally conceded that buildings of moderate height with solid masonry

construction, braced with permanent partitions, will require no special wind bracing; also that all steel-frame buildings in which the exterior walls are carried by the steel frame require some provision in the frame itself to enable it to resist the wind pressure.

The higher the building in proportion to its width, unless protected by adjacent buildings, the greater will be the need of efficient wind bracing.

The Chicago building ordinance makes the following requirements:

"In the case of all buildings the height of which is more than one and one half times their horizontal dimension, allowance shall be made for wind pressure, which shall not be figured at less than thirty pounds for each square foot of exposed surface. In buildings of skeleton construction the metal frame must be designed to withstand this wind pressure."

The building laws of Greater New York require that "All structures exposed to wind shall be designed to resist a horizontal wind pressure of thirty pounds for every square foot of surface thus exposed, from the ground to the top of the same, including, roof, in any direction." This is modified by the following clause: "In buildings under one hundred feet in height, provided the height does not exceed four times the average width of the base, the wind pressure may be disregarded."

The building laws of Boston and Philadelphia contain no reference to wind pressure. Mr. J. K. Freitag has a very practical chapter on wind bracing in his excellent work "Architectural Engineering."

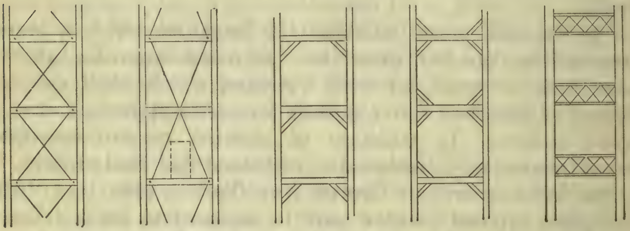
Methods of Wind Bracing.—For buildings not exceeding 120 ft. in height, and in which the least width is two thirds the height, sufficient rigidity will be obtained by using continuous column splices as in Figs. 15 and 18 of Chapter XIV, making the columns in two-story lengths, alternate columns breaking joint in alternate floors, and riveting both flanges of girders and beams to the columns by means of angles or brackets.

Exposed steel buildings in which the height exceeds one and one half times the width, or which are more than 120 ft. in height, should have some definite form of metallic bracing.

Fig. 3 shows in outline the different forms of wind bracing that have been employed to which should be added the "portal bracing" shown by Figs. 17 and 18. The form of bracing to

be employed in any given building will be governed by the peculiar conditions which the building offers

"The height, width, slope, and exposure of the structure, as well as the character of the enclosing walls, will determine the amount of wind pressure to be cared for, while the details of construction, the internal appearance, and the planning of the various floors will largely influence the manner in which the bracing is to be treated. The architectural planning of the offices, rooms, and corridors often raises most serious ob-



a b c d e

Fig. 3
Types of Wind Bracing.

stacles to a proper arrangement of wind bracing, and the engineer is frequently called upon to make most generous concessions for doors, windows, passages, and even whole areas, as is sometimes demanded in banking or assembly rooms and the like." (Freitag.)

"The bracing, whatever system is used, must, of course, be vertical, reaching down to some solid connection at the ground level. It should also be arranged in some regular symmetrical relation to the outlines of the building. For example, if the building is narrow and is braced crosswise with one system of bracing, that system should be midway between the ends of the building, and if two systems are used they should be equidistant from the ends, the exact distance being unimportant, because the floors when finished are extremely rigid. The symmetrical arrangement is necessary to secure an equal service of the systems and prevent any tendency to twist." (C. T. Purdy.)

Intensity of Wind Pressure.—The intensity of wind pressure which should be provided for in calculating the stresses in the braces, columns, and struts is considered at considerable length by Mr. Freitag. The building laws of New York and

Chicago specify a unit pressure of 30 lbs. Mr. Freitag thinks that 30 lbs. should serve as a minimum in high buildings of veneer construction.

It is seldom that the wind stresses are figured for a greater unit stress than 30 lbs.

Many engineers consider that fully 10 lbs. of wind pressure will be resisted by the connections between the columns and the floor system, partitions, and dead weight, so that if the bracing is computed to take care of 30 lbs. the building will be safe to resist an actual wind pressure of 40 lbs.*

Computation of Stresses.—As each different system of wind bracing creates stresses unlike those created by the other systems, each arrangement must be treated separately.

Diagonal Systems, or Sway-bracing.—The diagonal system of bracing shown by *a* and *b*, Fig. 3, is the cheapest and best when the division of the building by partitions will admit of its use.

The arrangement of diagonals shown at *a* is to be preferred, but the location of doors, etc., may sometimes be arranged to better advantage by making the rods pass through two stories, as shown at *b*. Sway-bracing was used in the Masonic Temple, the Venetian Building, and the Ashland Block in Chicago.

Analysis.—The wind stresses in a diagonal system are computed exactly as in Example 1, although as the posts are always vertical and the diagonals usually have the same inclination the stresses can readily be computed mathematically, as shown in the following example:

EXAMPLE 3.—Let Fig. 5 be an outline elevation of one set of bracing in a thirteen-story building having the same plan and horizontal dimensions as the Venetian Building, Fig. 4, and being protected on one side by an adjoining building which reaches to the sixth floor.

From an examination of the plan it will be seen that the exposed area contributory to each set of bracing for each story

* "The weight of the building affords some resistance, and in most cases is worth taking into account. Most buildings are filled with tile or some other sort of partitions, and when these are really constructed and their continuance is assured, there is no good reason why we should not rely also on them to some extent. There is also some resistance to lateral strains in the connection of the beams to the columns where they are well riveted. Some of these considerations will admit of calculation, but in using them much must depend on the experience and judgment of the engineer." (C. T. Purdy.)

is $21\frac{7}{12}' \times$ height of story, and as the stories are all 12 ft. from floor to floor, the area contributory to each joint is 259 sq. ft.

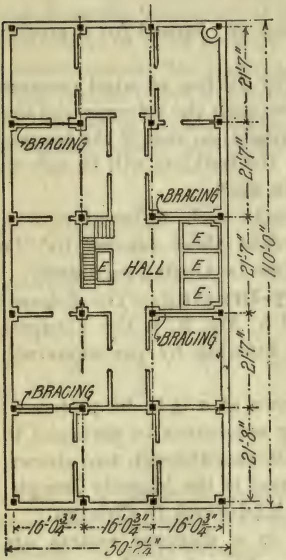


Fig. 4

Assuming a wind pressure of 30 lbs., the wind loads at each floor will be 7,770 lbs., or say 7,800 lbs., except that at the seventh floor the load will be only one half that at the floors above.

Note.—There will, of course, be a wind pressure on the stories below the seventh, but it is safe to assume that it will be resisted by the buildings abutting on the other side, particularly as the exposed side will, in the business portion of a city, be considerably sheltered by the buildings on the opposite side of the street.

In order to provide for door openings next the columns it will be necessary to connect the diagonals with the struts, 18″ in from centre of columns, which will make the angle between struts and diagonals 42° 42′.

(The tangent of the angle is $\frac{a}{b}$, or in this case .9230, and from the table on natural tangents, p. 122, we find the angle whose tangent is nearest to .9230 to be 42° 42′.)

We are not prepared to compute the stresses which should be entered in a table like that given below. As shown on p. 1079, the shear at each floor is equal to the wind load at that floor plus all of the loads above, which enables us to compute the shears directly and enter them in the second column. It was also shown that the stress in the diagonal for each story is equal to the shear in the strut above multiplied by the secant of the angle θ. From the table on p. 123*a* we find that the secant of 42° 42′ is 1.36, and multiplying the shears by this factor, we have the values given in the third column.

On p. 1079 it was shown that the stress in the leeward column for any story is equal to the stress in the story above plus the vertical component of the stress in the diagonal for that story,

and that the vertical component, to which we will give the term *increment*, is equal to the shear in the strut above multiplied by the tangent of the angle or by $\frac{a}{b}$, which in this example is .923.

Multiplying each shear by .923 we obtain the increments given in the fourth column of our table.

The compression in the leeward column in the thirteenth story is the same as the increment. In the twelfth story it is equal to the compression in the thirteenth story plus the increment for the twelfth story and so on down to the basement.

It will be seen that the compression in the columns increases very rapidly in the lower stories and amounts to a very considerable stress in the basement, first and second stories.

The tension in the windward column in any story is equal to the compression in the leeward column in the story next above, thus the tension in the windward column will be 471,600 lbs. in the first story and 525,600 lbs. in the basement.

The tensile stress exceeds the actual dead load on the columns, including a liberal allowance for weight of furniture, etc. In this example, however, the distance between the columns is very short in proportion to the height of the building, thus greatly increasing the column stress.

It is extremely doubtful if the wind blowing against a building such as we are considering would actually produce an uplift on the windward columns.

Theoretically both columns should be proportioned to the full dead load on the columns, including a small allowance for the weight of furniture, and also for the

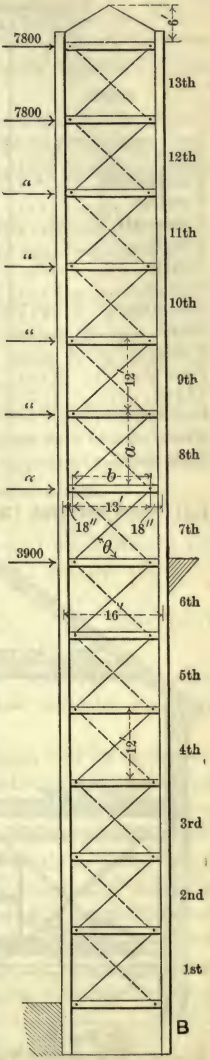


Fig. 5

WIND STRESSES. EXAMPLE 3.

Unit pressure = 30 lbs. Angle $\theta = 42° 42'$.
Tangent $\theta = 0.923$. Secant $\theta = 1.36$.

Story.	Shear or Compression in Strut.	Tension in Diagonal = shear × sec θ.	Increment = shear × tan θ.	Compression in Leeward Col.
Attic	7,800			
Thirteenth	15,600	10,600	7,200	7,200
Twelfth	23,400	21,220	14,400	21,600
Eleventh	31,200	31,820	21,600	43,200
Tenth	39,000	42,430	28,800	72,000
Ninth	46,800	53,040	36,000	108,000
Eighth	54,640	63,640	43,200	151,200
Seventh	58,500	74,250	50,400	201,600
Sixth	"	79,560	54,000	255,600
Fifth	"	"	"	309,600
Fourth	"	"	"	363,600
Third	"	"	"	417,600
Second	"	"	"	471,600
First	"	"	"	525,600
Basement	....	....	....	525,600

full wind stresses in the leeward column (as the wind may blow against either side of the building), and if the frame stood by itself, as in Fig. 5, this method should be followed in practice, and the columns should also be anchored to the foundations sufficient to resist the theoretical uplift minus the dead load.

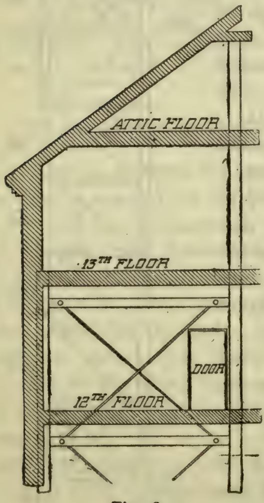


Fig. 6
Partial Cross-section, Venetian Building.

In the building under consideration, however, there are three of these panels in the width of the building (see Fig. 4), and as the floor connections would assist somewhat in relieving the braced system from the full theoretical stresses, most if not all structural engineers would probably cut down the theoretical stresses in the columns considerably, probably 50 per cent. in the lower stories.

The fact that buildings of even

more than thirteen stories are standing with scarcely any provision for wind stress * would indicate that a considerable cutting down of the column stresses is permissible.

Theoretically the columns should also be proportioned to the eccentric loads due to the increments being applied 18 ins. from the centre of the columns.

Practically the columns should be made wide, in the direction of the wind, i.e., parallel to the end of the building, and the braces should be applied as close to the centre of the columns as practical conditions will admit.

All columns affected by the wind bracing should be made continuous by means of splice plates from the foundation to the top.

Where the rods come down to the first-floor level, the bottom strut should be connected to the columns so as to take both tension and compression horizontally, so that both columns may assist in resisting the shear at that level.

The clearance spaces between all of the first-floor beams and columns should be filled with metal wedges and the columns wedged against the sidewalk walls, so that the entire floor system will act as a strut, backed by the solid street.

For the diagonal braces, which should run in both directions, square rods or flat bars should be used, and each should be provided with a turnbuckle for adjustment after the frame is erected.

It is customary to proportion the rods to the theoretical wind stresses, allowing 20,000 lbs. to the square inch.

The struts should be designed as strut beams if they also assist in supporting the floor, and for the bending moment produced by the bracing.

Fig. 7 shows the connection between column and strut recommended by Mr. Purdy. "The strut need not be connected to the column to resist horizontal forces, for there is no force tending to tear the strut away from the columns in this direction. The force to be resisted here is vertical. The strut should be made to butt the column squarely, instead of fastening to the sides of the column by rivets passing through the two members, or indirectly through connection plates, because the forces producing stresses in the bracing at this point must come into the strut by compression from without and not

* See Architectural Engineering, second edition p. 249.

through any possible tensile stress. When the strut butts the column these forces are introduced into the strut without the aid of rivets, and the full value of all the rivets can be used to

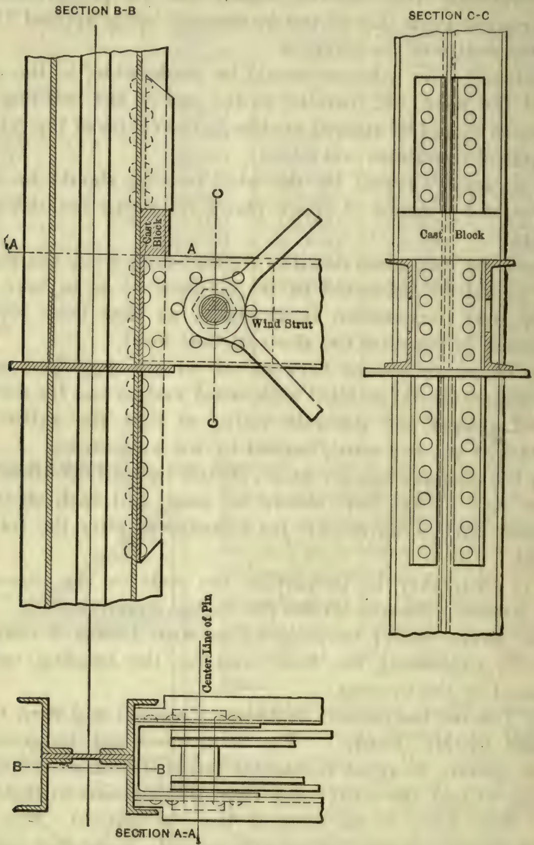


Fig. 7
Strut and Column Connections.

resist the vertical component of the rod stress. It serves also to keep the arm at the end of the strut, or the distance from the centre of pin to the bearing at the end, as short as possible, all of which is important. The top angles may be placed

several inches above the strut and a cast filler block introduced between them.

"Such an arrangement has several advantages. It generally happens that these angles cannot be riveted to the column directly above the channels of the strut, as shown in Fig. 7. The consequence is that whatever intervenes must carry a cross strain. The cast block will do this well. It is also important that there should be absolutely no clearance, otherwise the whole system would lack in stiffness and efficiency. The block can be cast a little large, and if necessary it can be chipped at the building in order to crowd it into position.

"The block also has the further advantage of cheapness and is always easily obtained. Every detail in wind bracing should receive the most careful attention." *

Fig. 8 shows a detail of the channel struts used in the Venetian Building up to and including the seventh floor. A lighter section was used for the floors above. These struts were independent of the floor system.

Knee Braces.—The system of wind bracing shown at *c*, Fig. 3, is not an economical method of bracing a framed structure, because it produces heavy bending moments both in the horizontal struts and in the columns. Nevertheless it is being

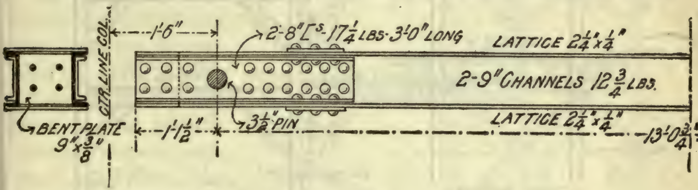


Fig. 8
Detail of Strut, Venetian Building.

used more largely in tall buildings than any other type of wind bracing, particularly for bracing the outer columns.

In a personal letter to the author, Mr. C. T. Purdy, whose firm (Purdy & Henderson) has been identified with the engineering work of a great many tall buildings built during the past ten years, says: "While it is true that gusset-plate and knee-brace construction is more expensive and not as desirable as the diagonal system, yet it is also true that we have used that

* C. T. Purdy in Modern Framed Structures, p. 459.

construction a good deal, but almost always in exterior walls. In all of these cases practical considerations have counted for more than theoretical ones. These practical considerations are: (1) On many buildings the arrangement or use of the building prevents the direct treatment of the problem and the owner or architect insists that the wind bracing shall be hidden

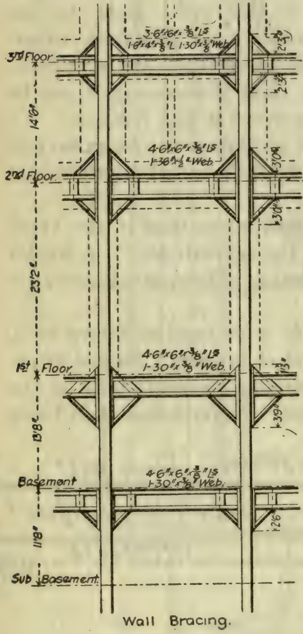


Fig. 9

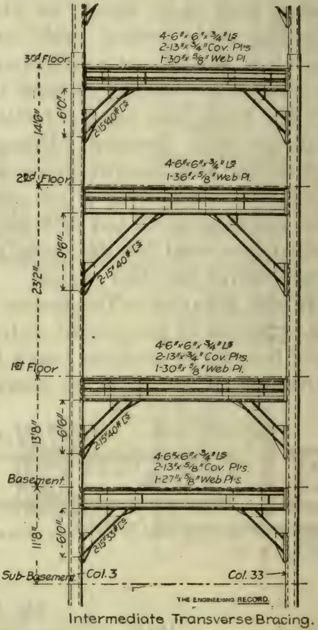


Fig. 10

Wind Bracing in Flat-iron Building.

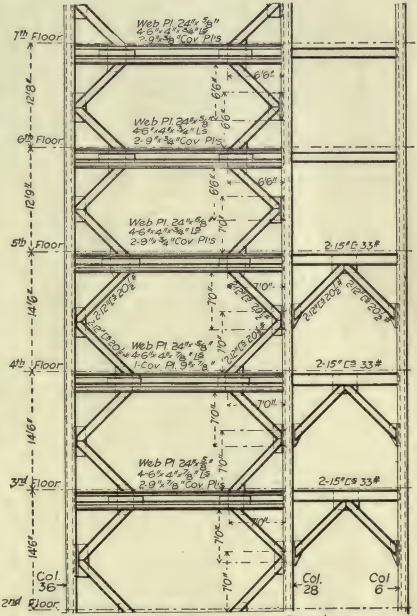
in the masonry regardless of cost. (2) When this construction is used we can make the heavy girder do double duty. In most cases the wall or floor construction also belonging to these particular members would require considerable metal and depth of brace. (3) Experience also shows that riveted construction in which all web members of a system can take either tension or compression makes the stiffest structure and is more satisfactory in every way than pin-connected work. In other words, although it costs a little more, the gusset-plate work

in exterior walls accomplishes its purpose and has proved very satisfactory."

Gusset plates were used in the Fort Dearborn Building, Chicago, by Jennie & Mundie, architects.

Figs. 9, 10, 11, and 12 * show details of the wind bracing in the Flat-iron Building, New York, of which D. H. Burnham & Co. were architects, Purdy & Henderson consulting engineers. This building is in plan a right-angled triangle with base and perpendicular 171 ft. and 86 ft. respectively, the angles of the building being curved, and the height is twenty-one stories, or about 285 ft. above the curb.

In all the outer walls of the building the masonry is carried on plate girders in each floor from the first to the twelfth stories respectively, and also at the eighteenth floor. All other stories above the twelfth have wall girders made of a pair of 15-in. channels.



Center Transverse Bracing.

Fig. 11

These wall girders are also utilized as wind struts and to support the floor beams. In all cases they are connected to the columns by solid web knee braces above and below the girder as shown by Figs. 9 and 14. Besides the knee bracing of the outside walls there are two systems of transverse bracing, shown in part by Figs. 10 and 11, connecting the two sides of the building. The system shown by Fig. 10 connects the

* These illustrations and the following description are from the *Engineering Record* of March 29, 1902.

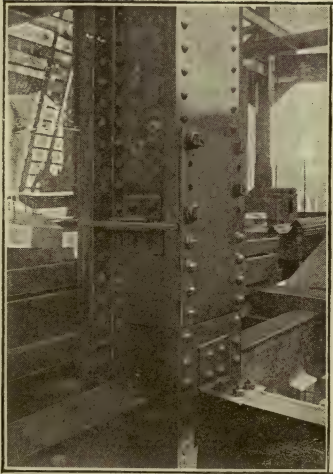


Fig. 12
Column Splice.

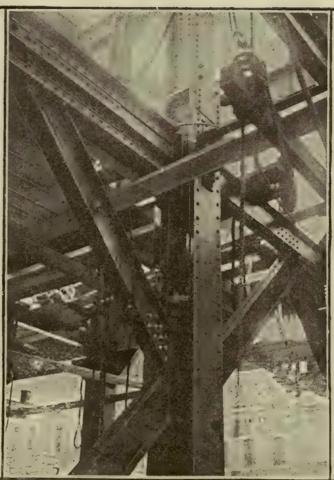


Fig. 13
Knee Bracing.

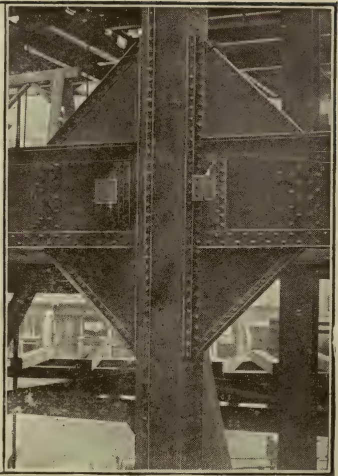


Fig. 14
Gusset Plates.

In the Flat-iron Building, New York.

third columns from the apex of the triangle, and that shown by Fig. 11 is 51 ft. or 3 panels beyond. "In addition to these two general systems of transverse bracing there is intermediate between them a supplemental system parallel with them, extending from the second floor to the foundation."

Figs. 13 and 14 are photographic views of the bracing, and Fig. 12 shows the manner in which the columns are spliced. The lower section of column 23 has a sectional area of 226.6 sq. ins.

The Frick Building, Pittsburg, a twenty-story steel-cage office building, is braced by plate girders and knee braces similar to those in the Flat-iron Building. A description of this building is contained in the *Engineering Record* of Jan. 11, 1902.

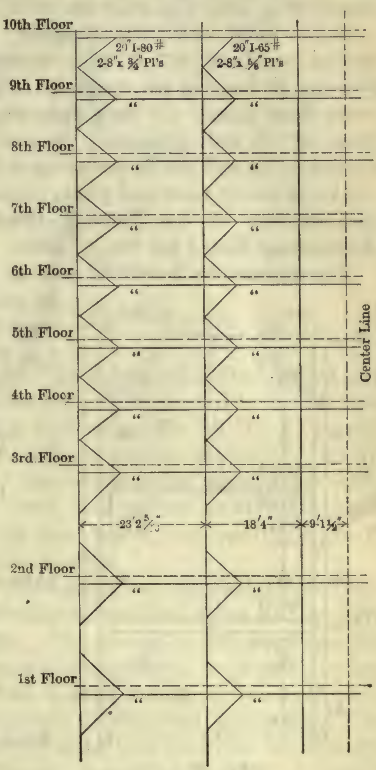


Fig. 15
Partial Transverse Section, Bank of the State of New York.

The building of the Bank of the State of New York, New York City,* which is about 85×100 ft. in plan and twenty-five stories, or about 340 ft. above the curb, has all four outside walls braced with long knee braces and also two rows of interior columns. Fig. 15 shows the bracing in two of the five panels in a section parallel to the front.

Details giving sizes of girders, bracing of exterior walls, etc., were published in the *Engineering Record* for Sept. 13, 1902.

* Clinton & Russell, architects; Purdy & Henderson, consulting engineers.

The Battery Place Building, New York,* has the two narrow ends of the building braced by struts formed of pairs of channels with solid web knee braces above and below, as in Fig. 9 (see *Engineering Record* of July 19, 1902).

In the Land and Title Building, Philadelphia, twenty-two stories, or about 317 ft. high above the curb, four different systems of bracing are combined: (1) horizontal diagonals in every floor plane; (2) deep plate girders at floor levels in the plane of all wall columns; (3) solid web knee braces in the corners of all wall panels excepting in the two upper stories; and (4) extra heavy beam and girder connections to interior columns (construction of the building illustrated and described in *Engineering Record* for Oct. 3, 1903).

The stresses for a system such as is shown in Fig. 16 may be computed with sufficient accuracy as follows: †

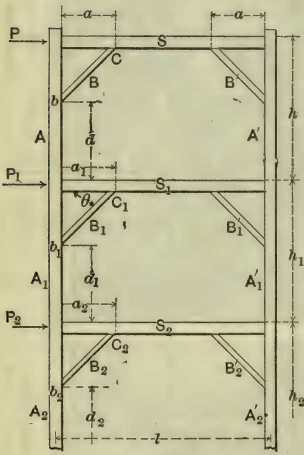


Fig. 16

Let P be the wind pressure at top floor, contributory to the bent;

P_1 the wind pressure at next floor below;

P_2 the wind pressure at second floor from top, and so on;

then max. compression in strut $s = P$;

max. compression in strut $s_1 = P + P_1$;

max. compression in strut $s_2 = P + P_1 + P_2$.

Tension in col. A = compression in $A' = V = \frac{P \times h}{l}$.

Increment for A_1 and $A_1' = V_1 = \frac{P + P_1 \times h_1}{l}$.

" " A_2 and $A_2' = V_2 = \frac{P + P_1 + P_2 \times h_2}{l}$.

---

* H. J. Hardenbergh, architect; Purdy & Henderson, consulting engineers.

† The best analysis of the stresses in braced portals and transverse bents that the author has seen is in "Steel Mill Buildings," by Prof. Milo S. Ketchum, C.E.

Tension in A_1 = compression in $A_1' = V + V_1$.

" " A_2 = " " $A_2' = V + V_1 + V_2$.

" " brace B = compression in $B' = V \sec \theta$.

" " " B_1 = " " $B_1' = V_1 \sec \theta$.

" " " B_2 = " " $B_2' = V_2 \sec \theta$.

Bending moment at $b = \dfrac{P + d}{4}$.

" " " $b_1 = \dfrac{P + P_1 \times d_1}{4}$.

" " " $b_2 = \dfrac{P + P_1 + P_2 \times d_2}{4}$.

" " " $c = \dfrac{P}{2} \times h - V \times a$.

" " " $c_1 = \dfrac{P + P_1}{2} \times h_1 - V_1 \times a$.

" " " $c_2 = \dfrac{P + P_1 + P_2}{2} \times h_2 - V_2 \times a$.

The struts s, s_1, s_2, etc., will be in tension at the leeward end; therefore both ends of the strut should be riveted to the columns.

In the Isabella Building in Chicago, Mr. W. L. B. Jenney, the architect, used the knee-brace system shown by Fig. 17. When the braces meet at the centre of the strut, there is no bending moment on the strut, and as the foot of the brace naturally comes nearer the floor below, the bending moment on the column is materially reduced.

Portal Bracing.—This system can be used in the place of sway-rods, where conditions as to corridors, doors, etc., prohibit the crossing of such spaces. The system is not as economical as sway-bracing, but is generally considered more effective and cheaper than the knee-brace system, because it produces practically no bending moment on the columns and struts. The portal system with a curved solid web was first used in the Old Colony Building, Chicago, completed in 1894, a partial cross-section of which is shown by Fig. 18 and a detail of one of the portals by Fig. 19.* "This arrangement of wind bracing proved very satisfactory in all respects."

It has since been used in a few panels in other buildings, but the knee brace seems generally to be preferred.

Analysis of Stresses.†—Fig. 20:

* From *Architectural Engineering.*

† The following analysis by Mr. C. T. Purdy is taken by permission from "Modern Framed Structures."

Let A = accumulated force or horizontal shear from wind at the floor next above floor M, applied half on one side and half on the other;

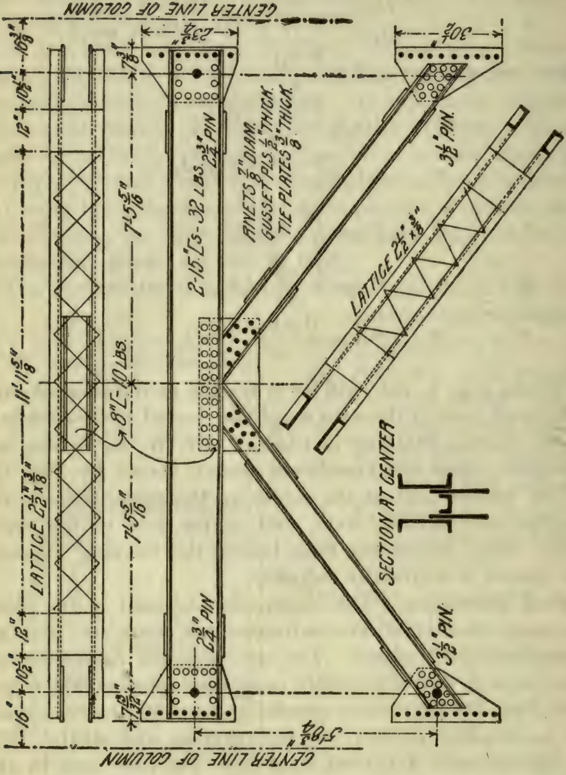


Fig. 17
Detail of Knee-bracing, Isabella Building.

B = the force of wind or shear directly tributary to floor M;

D = the accumulated vertical wind load in the column next above col. 2;

then

$\dfrac{Ah + Bh - Bc}{l}$ = vertical resistance due to A and B, or the increment, as denoted in the preceding analyses;

and

$\dfrac{A + B}{2}$ = horizontal reactions due to A and B.

The actual wind load on col. 2 and the corresponding tension in col. 1 will $= \frac{Ah+Bh-Bc}{l} + D$.

The horizontal shear along the line $EE = A + B$.

The horizontal shear in either leg below the line $EE = \frac{1}{2}(A+B)$.

The vertical shear on all vertical planes $= \frac{Ah+Bh-Bc}{l}$.

The thickness of the web-plates must be determined by these shears. It should be noted that the connection to the columns must be equal to the whole vertical shear. The direct compression in the flange $s = \frac{1}{2}B$.

Taking moments about the point of intersection of flange r with the line ww, it will be found that the sum of the moments = zero, that is, that there is no bending moment in the portal on the line ww and that flange t is not strained at this point.

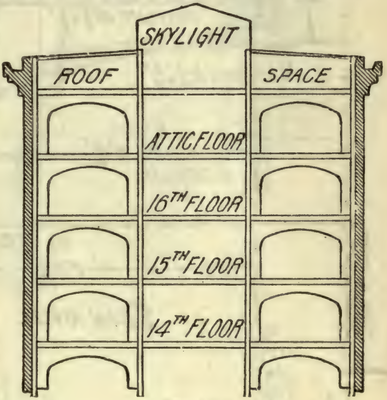


Fig. 18

Cross-section showing Portals, Old Colony Building.

For maximum stress in flange t take a point p in flange r, distant x from the line ww and at right angles to any given section of the flange t; then x times the vertical shear divided by y = the stress at the section taken, and this is maximum when $\frac{x}{y}$ has its greatest value.

The leg of the portal, including col. 2, might be also taken as a cantilever with two forces acting on it, $\frac{A+B}{2}$ and $\frac{Ah+Bh-Bc}{l}$, with flange t in compression and the column itself acting as a tension cord. Take a point in the centre of the column, distant x_1 from the bottom of the leg and at right angles to any given section in flange t, then $\frac{A+B}{2} \times \frac{x_1}{y_1}$ = the strain in flange t, and this is maximum when $\frac{x_1}{y_1}$ has its greatest value. There is

a slight error in this treatment, but it is on the side of safety. If flange t has a section proportioned to these maximum stresses, the requirements will be fulfilled. The stress and area re-

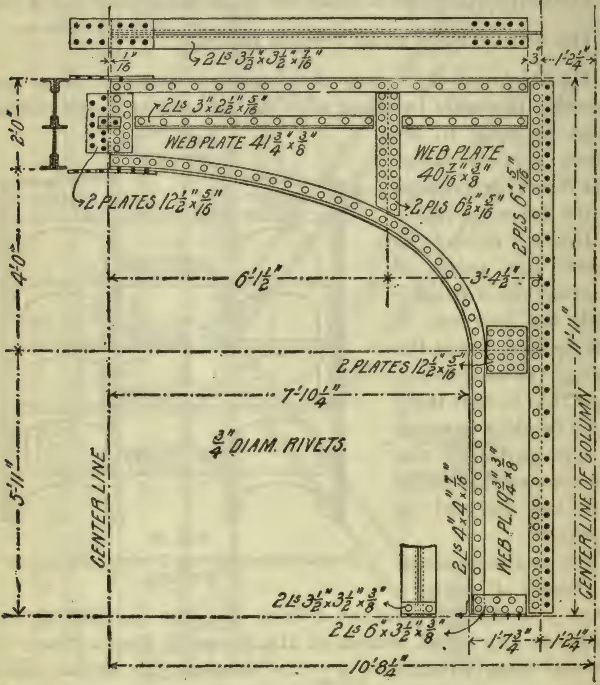


Fig. 19

Detail of Portal in Old Colony Building.

quired in flange r can be obtained in a similar manner. The connection of the portal above this flange to the portal and column above must be equal to $\frac{1}{2}A$ at each leg.

Lattice Girders.—"In this type of bracing the wind stresses are transferred to the ground on what is often called the 'table-leg principle;' that is, each story is made rigid in itself, the columns being figured as vertical beams to resist the lateral flexure due to the wind forces."

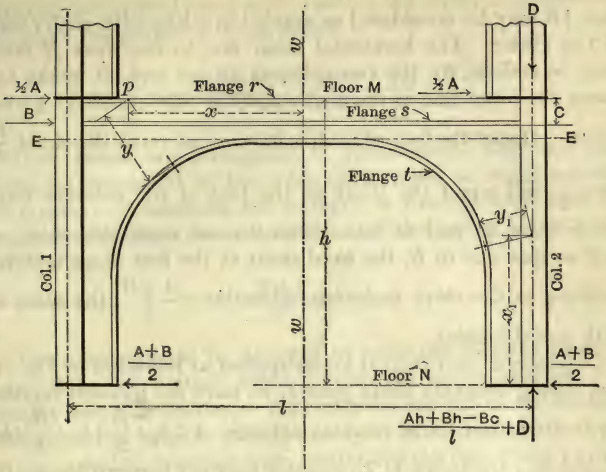


Fig. 20

Analysis.—Referring to Fig. 21:

Let A = accumulated force or horizontal shear from wind at the floor next above floor M, applied half on one column and half on the other;

B = the force of wind or shear directly tributary to floor M;

D = the accumulated vertical wind load in the column next above col. 2;

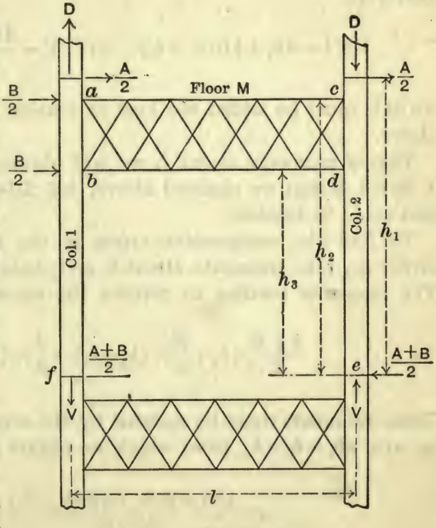


Fig. 21

then $\frac{1}{2}B$ may be considered as applied in a line with each chord of the girder. The horizontal shear due to the force B must then be resisted by the two columns at any and all points between the lower line of the girder and the top line of the girder below. Hence the foot of each column must resist the shear $\frac{B}{2}$. Also $\frac{A}{2}$ will equal the shear at the foot of the columns next above floor M, and as cols. 1 and 2 must resist this shear, as well as that due to B, the total shear at the foot of each of the columns in the story under consideration $= \frac{A+B}{2}$, the same as with portal bracing.

Considering the external forces applied as indicated in Fig. 21, and taking moments about joint e, we have the moment tending to overturn the frame on e as a hinge, $A \times h_1 + \frac{B}{2} \times h_2 + \frac{B}{2} \times h_3$. [Neglecting the forces D, D, because they act through the centre of the columns and do not affect the stresses in the girder or the bending moment in the column.] This moment must be resisted by the tension in col. 1, which acts with an arm l; therefore

$$V \times l = Ah_1 + \tfrac{1}{2}B(h_2 + h_3), \quad \text{or} \quad V = \frac{Ah_1 + \frac{1}{2}B(h_2 + h_3)}{l}.$$

To this must be added the load or tension D_1 from the column above.

Taking moments about f, we will obtain the same value for V in col. 2 that we obained above, col. 2 being in compression and col. 1 in tension.

To find the compressive stress in the upper flange of the girder ac, take moments about b and denote stress in ac by s. The moments tending to revolve the column to the right are

$$\frac{A+B}{2} \times h_3 + \frac{B}{2} \times (h_2 - h_3) + \frac{A}{2} \times (h_1 - h_3).$$

These moments must be resisted by the stress in ac acting with an arm ab, $= h_2 - h_3$, from which we obtain the equation

$$s = \frac{\frac{1}{2}(A+B)h_3 + \frac{1}{2}A(h_1 - h_3)}{h_2 - h_3} + \frac{B}{2}.$$

Taking moments about a and denoting the stress in the bottom flange of the girder by s_1 we obtain

$$s_1 = \frac{\frac{1}{2}(A+B)h_2 + \frac{1}{2}A(h - h_2)}{h_2 - h_3} - \frac{B}{2}.$$

Both struts are principally in compression, although they must be connected to the columns to resist an equal amount of tension. Considering the columns as fixed at both ends the maximum bending moments will be at the points b and d and will be equal to*

$$\frac{A+B}{2} \times \frac{1}{2}h_3.$$

The columns must be designed to resist this bending moment as well as the vertical loads. From the above analysis it is readily seen that the deeper the girder the less will be the stresses. When used in outside walls they should be made the full depth of the spandrel, reaching from just above the top of one window to immediately below the sills of the windows in the next story above.

* See Architectural Engineering, p. 277.

PART III.

USEFUL INFORMATION

FOR

ARCHITECTS, BUILDERS, AND SUPERINTENDENTS

AND ALL WHO HAVE TO DO WITH THE BUILDING TRADES.

NOTE.—The Author has endeavored to arrange the information herein contained in the following order:

Heating, Ventilation, Chimneys.
Hydraulics and Plumbing.
Illuminating-gas, Gas-piping, and Lighting.
Electrical Definitions, Rules, and Tables.
Weights, Quantities, and Data for Estimating Cost.
Dimensions and Data Useful in the Preparation of Plans.
Miscellaneous Information.
Glossary of Technical Terms.
Legal Definitions.

HEATING AND VENTILATION.

HEAT, FUEL, WATER, STEAM, AND AIR.

Heat is measured in two ways: 1st, by the thermometer, as in ordinary practice, and, 2d, by the work which it performs.

The *unit of heat* (sometimes called the British thermal unit) is that quantity of heat which will raise the temperature of one pound of water at or near the freezing-point, 1° Fahrenheit.

A French "*calorie*" is the heat required to raise one kilogramme of water 1° Centigrade, and is equal to 3.96832 British thermal units.

The equivalent in force of the unit of heat is the raising of 772 pounds avoirdupois one foot high, and is called the *mechanical equivalent of heat.*

Various kinds of fuel contain a certain number of thermal units per pound; and the method of heating which will convey the largest number of units to the air to be warmed is the most economical, so far as fuel and heating are concerned. But no method has yet been devised which will utilize more than about 85 per cent. of the heat-units contained in the fuel.

Fuel.*—The value of any fuel is measured by the number of heat-units which its combustion will generate. The fuels generally used in heating are composed of carbon and hydrogen, and ash, with sometimes small quantities of other substances not materially affecting its value.

"Combustible" is that portion which will burn, the ash or residue varying from 2 to 36 per cent. in different fuels.

The following table gives, for the more common combustibles, the air required for complete combustion, the temperature with different proportions of air, the theoretical value, and the highest attainable value under a steam-boiler, assuming that the gases pass off at 320°, the temperature of steam at 75 lbs. pressure, and the incoming draught to be at 60°.

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* From *Steam*, published by the Babcock & Wilcox Company, New York and Glasgow.

TABLE OF COMBUSTIBLES.

Kind of Combustible.	Air Required.	Temperature of Combustion.				Theoretical Value.		Highest Attainable Value under Boiler.	
	In Pounds per Pound of Combustible.	With Theoretical Supply of Air.	With 1½ Times the Theoretical Supply of Air.	With Twice the Theoretical Supply of Air.	With Three Times the Theoretical Supply of Air.	In Heat-units per Pound of Combustible.	In Pounds of Water Evaporated from and at 212° with 1 Pound Combustible.	With Chimney Draft.	With Blast, Theoretical Supply of Air at 60° Gas 320°.
Hydrogen	36.00	5750	3860	2860	1940	62032	64.20		
Petroleum	15.43	5050	3515	2710	1850	21000	21.74	18.55	19.90
Carbon { Charcoal, Coke, Anthracite coal }	12.13	4580	3215	2440	1650	14500	15.00	13.30	14.14
Coal—Cumberland	12.06	4900	3360	2550	1730	15370	15.90	14.28	15.06
" Coking bituminous	11.73	5140	3520	2680	1810	15837	16.00	14.45	15.19
" Cannel	11.80	4850	3330	2540	1720	15080	15.60	14.01	14.76
" Lignite	9.30	4600	3210	2490	1670	11745	12.15	10.78	11.46
Peat—Kiln-dried	7.68	4470	3140	2420	1660	9660	10.00	8.92	9.42
" { Air-dried, 25 per cent. water }	5.76	4000	2820	2240	1550	7000	7.25	6.41	6.78
Wood—Kiln-dried	6.00	4080	2910	2260	1530	7245	7.50	6.64	7.02
" { Air-dried, 20 per cent. water }	4.80	3700	2670	2100	1490	5600	5.80	4.08	4.39

The effective value of all kinds of wood *per pound*, when dry, is substantially the same. The following are the weights and comparative value of different woods *by the cord:*

Kind of Wood.	Weight.	Kind of Wood.	Weight.
Hickory, shellbark	4469	Southern pine	3375
Hickory, red-heart	3705	Virginia pine	2680
White oak	3821	Spruce	2325
Red oak	3254	New Jersey pine	2137
Beech	3126	Yellow pine	1904
Hard maple	2878	White pine	1868

The following table of American coals has been compiled from various sources:

AMERICAN COALS.

Coal. State.	Coal. Kind of Coal.	Per Cent. of Ash.	Theoretical Value. In Heat-units.	Theoretical Value. In Pounds of Water Evaporated.
Pennsylvania,	Anthracite......	3.49	14,199	14.70
"	"	6.13	13,535	14.01
"	"	2.90	14,221	14.72
"	Cannel.........	15.02	13,143	13.60
"	Connellsville....	6.50	13,368	13.84
"	Semi-bituminous	10.77	13,155	13.62
"	Stone's gas.....	5.00	14,021	14.51
"	Youghiogheny...	5.60	14,265	14.76
"	Brown.........	9.50	12,324	12.75
Kentucky,	Caking.........	2.75	14,391	14.89
"	Cannel.........	2.00	15,198	16.76
"	"	14.80	13,360	13.84
"	Lignite.........	7.00	9,326	9.65
Illinois,	Bureau Co......	5.20	13,025	13.48
"	Mercer Co......	5.60	13,123	13.58
"	Montauk.......	5.50	12,659	13.10
Indiana,	Block..........	2.50	13,588	14.38
"	Caking.........	5.66	14,146	14.64
"	Cannel.........	6.00	13,097	13.56
Maryland,	Cumberland....	13.98	12,226	12.65
Arkansas,	Lignite.........	5.00	9,215	9.54
Colorado,	"	9.25	13,562	14.04
"	"	4.50	13,866	14.35
Texas,	"	4.50	12,962	13.41
Washington,	"	3.40	11,551	11.96
Pennsylvania,	Petroleum......	....	20,746	21.47

"Slack," or the screenings from coal, when properly mixed—anthracite and bituminous—and burned by means of a blower on a grate adapted to it, is nearly equal in value of combustible to coal, but its percentage of refuse is greater.

One pound of *pure carbon*, when completely burned, yields 14,500 heat-units.

Temperature of Fire.

By reference to the table of combustibles it will be seen that the temperature of the fire is nearly the same for all kinds of combustibles under similar conditions. If the temperature

is known, the conditions of combustion may be inferred. The following table, from M. Pouillet, will enable the temperature to be judged by the appearance of the fire:

Appearance.	Temperature F.	Appearance.	Temperature F.
Red, just visible	977°	Orange, deep	2010°
" dull	1290°	" clear	2190°
" cherry dull	1470°	White heat	2370°
" " full	1650°	" bright	2550°
" " clear	1830°	" dazzling	2730°

To determine temperature by fusion of metals, etc.

The following figures for the melting-points of various substances are given by Clark (on the authority of Pouillet, Claudel, and Wilson), except those marked †, which are given by Prof. Roberts-Austen in his description of the Le Chatelier pyrometer. These latter are probably the most reliable figures.*

	Deg. F.		Deg. F.
Mercury	−39	Antimony	810 to 1150
Ice	32	Aluminium	1157†
Tallow	92	Magnesium	1200
Stearine	109 to 120	Calcium	Full red heat
Spermaceti	120	Bronze	1692
Wax	142 to 154	Silver	1733† to 1873
Sodium	194 to 208	Potassium sulphate	1859†
Alloy, 3 lead, 2 tin, 5 bismuth	199	Gold	1913† to 2282
Sulphur	239	Copper	1929† to 1996
Alloy, 1½ tin, 1 lead	334	Cast iron, white	1922 to 2075†
Alloy, 1 tin, 1 lead	370 to 466	" " gray	2012 to 2786 2228†
Tin	442 to 446	Steel	2372 to 2532
Cadmium	442	" hard	2570†; mild, 2687†
Bismuth	504 to 507	Wrought iron	2732 to 2912
Lead	608 to 618†	Palladium	2732†
Zinc	680 to 779†	Platinum	3227†

Water.—The several conditions of water are usually stated as the solid, the liquid, and the gaseous. Two conditions are covered by the last term, and water should be understood as capable of existing in four different conditions—the solid, the liquid, the vaporous, and the gaseous.

At and below 32° F. water exists in the solid state, as ice; at 39° F., it reaches its maximum density. Above 39° the density diminishes.

The weight per cubic foot of water at different temperatures and under pressure of one atmosphere is shown to two places of decimals by the table on p. 1112 (calculated by Rankine's formula):

The boiling-point of water depends upon the pressure. Thus

* Kent, p. 455.

at one atmosphere (14 7 lbs., 29.22″ barometer) the temperature of ebullition is 212°. With a partial vacuum, or absolute pressure of 1 lb. (2.037″of mercury), the boiling-point is 101.40 F.

On the other hand, if the pressure be 74.7 lbs. absolute (60 lbs. by the gauge), the temperature of evaporation becomes 307° F.

When water is freed of air, it may be elevated in temperature to 270° before evaporation takes place.

Steam.

Dry steam is steam not containing any free moisture. It may be either saturated or superheated.

Wet steam is steam containing free moisture in the form of spray or mist, and has the same temperature as dry saturated steam of the same pressure.

Saturated steam is steam in its normal state, that is, steam whose temperature is that due its pressure; by which is meant steam at the same temperature as that of the water from which it was generated and upon which it rests.

Superheated Steam.—Steam which has a higher temperature than that normal to its pressure is termed "superheated," or "gaseous." Dr. Siemens found that when steam at 212° was heated *separate from water* it increased rapidly in volume, up to 230°, after which it expanded uniformly, as a permanent gas. The use in any steam-boiler of superheating surface exposed to the heated gases of combustion is highly objectionable and is of doubtful efficiency. Steam cannot be superheated when in contact with water.

Sensible and Latent Heat of Steam.—The temperature of steam, as shown by the thermometer, is called its sensible heat, and this varies with every different pressure; but it is found that steam contains more heat than is shown by the thermometer, and this extra heat is called the *latent heat* of steam.

The following table gives the number of British thermal units in a pound of water at different temperatures below the boiling-point. They are reckoned above 32° F.; for, strictly speaking, *water* does not exist below 32°, and ice follows another law.

When a solid becomes a liquid or a liquid becomes a vapor heat is absorbed, more than was necessary to raise it to the temperature of conversion; and this *latent heat* does work in

HEAT-UNITS IN WATER, BETWEEN 32° AND 212° F., AND WEIGHT OF WATER PER CUBIC FOOT.

Temp. Deg. Fahr.	Heat-units.	Weight, Lbs. per Cu. Ft.	Temp. Deg. Fahr.	Heat-units.	Weight, Lbs. per Cu Ft.	Temp. Deg. Fahr.	Heat-units.	Weight, Lbs. per Cu. Ft.
32	0	62.42	123	91.16	61.68	168	136.44	60.81
35	3	62.42	124	92.17	61.67	169	137.45	60.79
40	8	62.42	125	93.17	61.65	170	138.45	60.77
45	13	62.42	126	94.17	61.63	171	139.46	60.75
50	18	62.41	127	95.18	61.61	172	140.47	60.73
52	20	62.40	128	96.18	61.60	173	141.48	60.70
54	22.01	62.40	129	97.19	61.58	174	142.49	60.68
56	24.01	62.39	130	98.19	61.56	175	143.50	60.66
58	26.01	62.38	131	99.20	61.54	176	144.51	60.64
60	28.01	62.37	132	100.20	61.52	177	145.52	60.62
62	30.01	62.36	133	101.21	61.51	178	146.52	60.59
64	32.01	62.35	134	102.21	61.49	179	147.53	60.57
66	34.02	62.34	135	103.22	61.47	180	148.54	60.55
68	36.02	62.33	136	104.22	61.45	181	149.55	60.53
70	38.02	62.31	137	105.23	61.43	182	150.56	60.50
72	40.02	62.30	138	106.23	61.41	183	151.57	60.48
74	42.03	62.28	139	107.24	61.39	184	152.58	60.46
76	44.03	62.27	140	108.25	61.37	185	153.59	60.44
78	46.03	62.25	141	109.25	61.36	186	154.60	60.41
80	48.04	62.23	142	110.26	61.34	187	155.61	60.39
82	50.04	62.21	143	111.26	61.32	188	156.62	60.37
84	52.04	62.19	144	112.27	61.30	189	157.63	60.34
86	54.05	62.17	145	113.28	61.28	190	158.64	60.32
88	56.05	62.15	146	114.28	61.26	191	159.65	60.29
90	58.06	62.13	147	115.29	61.24	192	160.67	60.27
92	60.06	62.11	148	116.29	61.22	193	161.68	60.25
94	62.06	62.09	149	117.30	61.20	194	162.69	60.22
96	64.07	62.07	150	118.31	61.18	195	163.70	60.20
98	66.07	62.05	151	119.31	61.16	196	164.71	60.17
100	68.08	62.02	152	120.32	61.14	197	165.72	60.15
102	70.09	62.00	153	121.33	61.12	198	166.73	60.12
104	72.09	61.97	154	122.33	61.10	199	167.74	60.10
106	74.10	61.95	155	123.34	61.08	200	168.75	60.07
108	76.10	61.92	156	124.35	61.06	201	169.77	60.05
110	78.11	61.89	157	125.35	61.04	202	170.78	60.02
112	80.12	61.86	158	126.36	61.02	203	171.79	60.00
114	82.13	61.83	159	127.37	61.00	204	172.80	59.97
115	83.13	61.82	160	128.37	60.98	205	173.81	59.95
116	84.13	61.80	161	129.38	60.96	206	174.83	59.92
117	85.14	61.78	162	130.39	60.94	207	175.84	59.89
118	86.14	61.77	163	131.40	60.92	208	176.85	59.87
119	87.15	61.75	164	132.41	60.90	209	177.86	59.84
120	88.15	61.74	165	133.41	60.87	210	178.87	59.82
121	89.15	61.72	166	134.42	60.85	211	179.89	59.79
122	90.16	61.70	167	135.43	60.83	212	180.90	59.76

the destruction of the force of cohesion and other changes which take place, and must be absorbed from some other substance. In the case of steam in a boiler, it comes from the fuel during combustion. When steam or vapor is condensed, this same quantity of heat that was received, from whatever source, is again given off to any substance within its influence—air, water, iron pipes, etc.—colder than itself; and it is this property, together with its great power of absorbing and re-

taining heat, which makes water and its vapor such a valuable medium for conveying heat from the furnace to the rooms to be warmed.

The *specific heat* (or heat-absorbing capacity) of water is not constant, but rises in an increasing ratio with the temperature; so that it requires more heat the higher the temperature to raise a given quantity of water from one temperature to another. Thus, the specific heat at 32° being 1, at 212° it is 1.013, and at 320° (the temperature of 75 lbs. steam pressure) it is 1.0294.

The amount of heat *rendered latent* by each pound of water in becoming steam varies at different pressures, decreasing as the pressure increases. This latent heat, added to the sensible heat (or thermometric temperature), constitutes the "total heat." The "total heat" being greater as the pressure increases, it will take more heat, and consequently more fuel, to make a pound of steam the higher the pressure.

The table given on the following page shows the properties of steam at different pressures, from 1 lb. to 400 lbs. "total pressure"; i.e., above vacuum.

The gauge pressure is about 15 lbs. less than the total pressure; so that, in using this table, 15 must be added to the pressure as given by the steam-gauge.

The column of "Temperatures" gives the thermometric temperature of steam and boiling-point at each pressure.

The "Factor of Equivalent Evaporation" shows the proportionate cost, in heat or fuel, of producing steam at the given pressure, as compared with atmospheric pressure. To ascertain the equivalent evaporation at any pressure, multiply the given evaporation by the factor of its pressure and divide the quotient by the factor of the desired pressure.

Each degree of difference in temperature of feed-water makes a difference of 0.00104 in the amount of evaporation. Hence, to ascertain the equivalent evaporation from any other temperature of feed than 212°, add to the factor given as many times 0.00104 as the temperature of feed-water is degrees below 212°.

For other pressures than those given in the table, it will be practically correct to take the proportion of the difference between the nearest pressures given in the table.

Air.—Air is a mechanical mixture of *oxygen* and *nitrogen*, the proportion for pure air being 77 per cent. of nitrogen and 23 per cent. of oxygen by weight. It also contains about $\frac{1}{2000}$

TABLE OF PROPERTIES OF SATURATED STEAM.*

Total Pressure per Square Inch.	Temperature in Fahrenheit Degrees.	Total Heat in Heat-units from Water at 32° Fahr.	Latent Heat in Heat-units.	Density or Weight of 1 Cubic Foot.	Volume of One Pound of Steam.	Relative Volume or Cu. Ft. of Steam from 1 Cu. Ft. of Water.	Factor of Equivalent Evaporation from Water at 212°.
1	102	1113.05	1042.964	0.0030	330.36	20620	0.965
2	126.266	1120.45	1026.010	0.0058	172.08	10720	0.972
3	141.622	1125.131	1015.254	0.0085	117.52	7326	0.977
4	153.070	1128.625	1007.229	0.0112	89.62	5600	0.981
5	162.330	1131.449	1000.727	0.0137	72.66	4535	0.984
6	170.123	1133.826	995.249	0.0163	61.21	3814	0.986
7	176.910	1135.896	990.471	0.0189	52.94	3300	0.988
8	182.910	1137.726	986.245	0.0214	46.69	2910	0.990
9	188.316	1139.375	982.434	0.0239	41.79	2607	0.992
10	193.240	1140.877	978.958	0.0264	31.84	2360	0.994
15	213.025	1146.912	964.973	0.0387	25.85	1612	1.000
20	227.917	1151.454	954.415	0.0511	19.72	1220.3	1.005
25	240.000	1155.139	945.825	0.0634	15.99	984.8	1.008
30	250.245	1158.263	938.925	0.0755	13.46	826.8	1.012
35	259.176	1160.987	932.152	0.0875	11.65	713.4	1.015
40	267.120	1163.410	926.472	0.0994	10.27	628.2	1.017
45	274.296	1165.600	921.334	0.1111	9.18	561.8	1.017
50	280.854	1167.600	916.631	0.1227	8.31	508.5	1.021
55	286.897	1169.442	912.290	0.1343	7.61	464.7	1.023
60	292.520	1171.158	908.247	0.1457	7.01	428.5	1.025
65	297.777	1172.762	904.462	0.1569	6.49	397.7	1.027
70	302.718	1174.269	900.899	0.1681	6.07	371.2	1.028
75	307.388	1175.692	897.526	0.1792	5.68	348.3	1.030
80	311.812	1177.042	894.330	0.1901	5.35	328.3	1.031
85	316.021	1178.326	891.286	0.2010	5.05	310.5	1.033
90	320.039	1179.551	888.375	0.2118	4.79	294.7	1.034
95	323.884	1180.724	885.588	0.2224	4.55	280.6	1.035
100	327.571	1181.849	883.914	0.2330	4.33	267.9	1.036
105	331.113	1182.929	880.342	0.2434	4.14	265.5	1.037
110	334.523	1183.970	877.865	0.2537	3.97	246.0	1.038
115	337.814	1184.974	875.472	0.2640	3.80	236.3	1.039
120	340.995	1185.944	873.155	0.2742	3.65	227.6	1.040
125	344.074	1186.883	870.911	0.2842	3.51	219.7	1.041
130	347.059	1187.794	868.735	0.2942	3.38	212.3	1.042
140	352.757	1189.535	864.566	0.3138	3.16	199.0	1.044
150	358.161	1191.180	860.621	0.3340	2.96	187.5	1.046
160	363.277	1192.741	856.874	0.3520	2.79	177.3	1.047
170	368.158	1194.228	853.294	0.3709	2.63	168.4	1.049
180	372.822	1195.650	849.869	0.3889	2.49	160.4	1.051
190	377.291	1197.013	846.584	0.4072	2.37	153.4	1.052
200	381.573	1198.319	843.432	0.4249	2.26	147.1	1.053
250	401.072	1203.735	831.222	0.5464	1.83	114	1.059
300	418.225	1208.737	819.610	0.6486	1.54	96	1.064
350	431.956	1212.580	810.690	0.7498	1.33	83	1.068
400	444.919	1217.094	800.198	0.8502	1.18	73	1.073

of its volume of carbonic-acid gas and some watery vapor, and is capable of absorbing any other gas or vapor to a certain extent, distributing them through the whole atmosphere by

* *Steam*, 14th ed. Babcock & Wilcox Company, New York and Glasgow.

what is called *the law of diffusion of gases*, a property which gases have of mixing and diluting, which prevents gases of different specific gravities from stratifying for any considerable time. This property is of the utmost importance to air; for, if any noxious or poisonous gas were to remain separated in the atmosphere, any one breathing it would be instantly killed.

Air at 60° F., and with the barometer at 30 ins., is taken as the standard for the comparison of the weight of gases, itself being considered as unity.

At the temperature of 32°, $13\frac{1}{4}$ cu. ft. of air weigh a few grains over 1 lb. avoirdupois.

The expansion of air is nearly uniform at all temperatures, expanding about $\frac{1}{490}$ of its bulk at 32° and for each increase of *one degree* in temperature.

The table on the following page, giving the volume and weight of dry air, tension and weight of vapor, etc., will be found useful for reference. In this table 1,000 cu. ft. of dry air is taken for a unit, and the coefficient of expansion is taken at $\frac{1}{490}$, the air being under constant pressure of 30 ins. of mercury. Column 5 is taken from Guyot's tables, Regnault's data.

Watery Vapor in the Atmosphere.—Air is capable of holding or absorbing a certain quantity of vapor of water, the proportion depending on the temperature of the air.

The warmer it is the larger quantity it will hold, and as it becomes cool again, it deposits it, or forms clouds or fogs which condense on anything colder than the air, leaving the air, upon raising its temperature, capable of taking up more moisture, to be again deposited in dew or rain. It is this property of air which gives it its drying qualities.

An absolutely dry atmosphere is almost an impossibility. Air at 32° contains, when saturated with moisture, $\frac{1}{160}$ of its weight of water; at 59° it contains $\frac{1}{80}$; at 86° it contains $\frac{1}{40}$; its capacity for moisture being doubled by each increase of 27° F.

Air is said to be "saturated" when it has absorbed all the water it will hold at that temperature. The *tension* of vapors is the elastic force or pressure which they exert on the sides of vessels in which they are contained.

Air, to be healthful, should contain about 75 per cent. of the moisture required for saturation.

It requires more heat to raise the temperature of a given quantity of moist air 1° than for dry air; but unless the air is saturated this difference is not of much practical importance.

VOLUME AND WEIGHT OF AIR AND WEIGHT OF VAPOR IN SATURATED AIR.

Temperature.	Volume.	Number of Cubic Feet to 1 Pound.	Weight of 1,000 Cubic Feet Dry Air.	Tension of Vapor.	Weight of Vapor Saturated in 1,000 Cubic Feet.	W'ght of Air Displaced by Vapor.
1	2	3	4	5	6	7
0	0.9340	11.460	87.260	0.04379	0.07930	0.1264
5	0.9449	11.591	86.289	0.05747	0.10289	0.1646
10	0.9551	11.726	85.251	0.07116	0.12588	0.2014
15	0.9653	11.869	84.317	0.08535	0.14932	0.2389
20	0.9755	11.992	88.403	0.10748	0.18180	0.2909
25	0.9857	12.125	82.440	0.13367	0.22871	0.3661
30	0.9959	12.258	81.566	0.16581	0.27491	0.4398
32	1.0000	12.311	81.235	0.17989	0.29633	0.4741
36	1.0082	12.417	80.515	0.21066	0.35201	0.5632
40	1.0163	12.523	79.872	0.24604	0.40770	0.6523
44	1.0244	12.629	79.176	0.28647	0.47070	0.7531
48	1.0326	12.735	78.493	0.33284	0.54204	0.8672
52	1.0408	12.841	77.825	0.38574	0.62282	0.9965
56	1.0489	12.947	77.220	0.44352	0.71063	1.1370
60	1.0571	13.053	76.628	0.51683	0.82173	1.3147
64	1.0652	13.159	75.988	0.59229	0.93390	1.4943
68	1.0734	13.265	75.357	0.67994	1.0631	1.7008
72	1.0816	13.371	74.794	0.78018	1.21050	1.9368
76	1.0897	13.477	74.184	0.89103	1.31715	2.1076
80	1.0979	13.583	73.638	1.01669	1.5540	2.4864
84	1.1060	13.689	73.046	1.15705	1.7536	2.8058
88	1.1142	13.795	72.464	1.31554	1.9772	3.1635
92	1.1223	13.901	71.942	1.49067	2.2257	3.5611
96	1.1305	14.007	71.377	1.69214	2.5060	4.0096
100	1.1387	14.113	70.872	1.91937	2.8220	4.5152
104	1.1468	14.219	70.323	2.14669	3.133	5.0138
108	1.1550	14.325	69.784	2.43323	3.523	5.6368
112	1.1631	14.431	69.300	2.72984	3.926	6.2826
116	1.1713	14.537	68.776	3.05954	4.367	6.9882
120	1.1794	14.643	68.306	3.41728	4.843	7.7488
124	1.1876	14.749	67.797	3.81775	5.371	8.5940
128	1.1957	14.855	67.295	4.26073	6.088	9.7430
132	1.2039	14.961	66.845	4.72888	6.559	10.4950
136	1.2121	15.067	66.357	5.25807	7.240	11.584
140	1.2202	15.173	65.919	5.81736	7.957	12.731
144	1.2284	15.279	65.442	6.48029	8.800	14.048
148	1.2365	15.385	64.977	7.14323	9.630	15.408
152	1.2447	15.491	64.568	7.9104	10.595	16.952
156	1.2528	15.597	64.102	8.6923	11.566	18.506
160	1.2610	15.703	63.694	9.5948	12.681	20.290
164	1.2691	15.809	63.251	10.5579	13.828	22.125
168	1.2773	15.915	62.814	11.4673	14.950	23.920
172	1.2855	16.021	62.422	12.7165	16.47	26.36
176	1.2936	16.127	61.996	13.8657	17.43	27.89
180	1.3018	16.233	61.614	15.2343	19.47	31.96
184	1.3099	16.339	61.200	16.6030	21.08	33.73
188	1.3181	16.445	60.790	18.1447	22.89	36.63
192	1.3262	16.551	60.423	19.7441	24.75	39.60
196	1.3344	16.657	60.024	21.4297	26.69	42.71
200	1.3426	16.763	59.666	23.2962	28.85	46.16

Columns 6 and 7 of the above table give the weight of vapor in 1,000 cu. ft. of saturated air, and the weight of displaced air, for different temperatures from 0° to 200°.

The numbers in column 6 are obtained by multiplying the corresponding numbers in column 4 by column 5 and the product by $\frac{6225}{30}$. Column 7 is obtained from column 6 by multiplying the values in column 6 by $\frac{8}{5}$.

Specific Heat of Air.—The specific heat of any substance is the quantity of heat required to raise its temperature 1° compared with the quantity of heat required to raise the temperature of 1 lb. of water at the same temperature 1°. The specific heat of air, as determined by Regnault, is 0.2374. Hence one thermal unit will raise the temperature of 1 lb. of water or $4\frac{1}{5}$ lbs. of dry air (equals 51.7 cu. ft. at 32° F.) 1° F. As all air contains more or less moisture, which must also be warmed, 50 cu. ft. is generally considered as the equivalent of 1 lb. of water in heating.

As 1 lb. of steam at 0 (gauge) pressure condensed to water gives off 965 thermal units, it is therefore equivalent to warming about 48,000 cu. ft. of air 1°.

Drying by Steam.*

There are three modes of drying by steam: 1st, by bringing wet substances in direct contact with steam-heated surfaces, as by passing cloth or paper over steam-heated cylinders, or clamping veneers between steam-heated plates; 2d, by radiated heat from steam-pipes, as in some lumber-kilns and laundry drying-rooms; 3d, by causing steam-heated air to pass over wet surfaces, as in glue-works, etc.

The second is rarely used except in combination with the third.

The first is most economical, the second less so, and the third least. Under favorable circumstances it may be estimated that 1 horse-power of steam will evaporate 24 lbs. of water by the first method, 20 by the second, and 15 by the third.

The philosophy of drying or evaporating moisture by heated air rests upon the fact that the capacity of air for moisture is rapidly increased by rise in temperature. If air at 52° is heated to 72°, its capacity for moisture is doubled, and is four times what it was at 32°. The following table gives the weight of a saturated mixture of air and aqueous vapor at different temperatures up to 160°, the practical limit of heating air by steam, together with the weight of vapor in pounds and percentage, and total heat, with the portion thereof contained in the vapor:

* From *Steam*. Babcock & Wilcox Company.

SATURATED MIXTURES OF AIR AND AQUEOUS VAPOR.

Temperature, Degrees Fahrenheit.	Weight of 100 Cu. Ft. of Mixture, in Pounds.	Weight of Water in 100 Cu. Ft. of Mixture, in Pounds.	Per Cent. of Water in Mixture.	Heat-units in 100 Cu. Ft. of Mixture.	Per Cent. of Heat in Vapor.	Temperature, Degrees Fahrenheit.	Weight of 100 Cu. Ft. of Mixture, in Pounds.	Weight of Water in 100 Cu. Ft. of Mixture in Pounds.	Per Cent. of Water in Mixture.	Heat-units in 100 Cu. Ft. of Mixture.	Per Cent. of Heat in Vapor.
35	8.004	0.034	0.42	42.8	86.69	100	6.924	0.283	4.08	422.0	74.58
40	7.920	0.041	0.52	59.8	76.59	105	6.830	0.325	4.76	474.7	76.22
45	7.834	0.049	0.62	77.7	68.98	110	6.741	0.373	5.23	533.9	77.88
50	7.752	0.059	0.76	97.6	66.29	115	6.650	0.426	6.41	599.1	79.52
55	7.688	0.070	0.91	118.3	64.58	120	6.551	0.488	7.46	672.4	81.14
60	7.589	0.082	1.08	140.1	64.31	125	6.454	0.554	8.55	750.5	82.62
65	7.507	0.097	1.29	164.9	64.76	130	6.347	0.630	9.90	839.4	84.13
70	7.425	0.114	1.49	189.7	66.21	135	6.238	0.714	11.44	936.7	85.57
75	7.342	0.134	1.79	221.6	66.74	140	6.131	0.806	13.14	1042.7	86.89
80	7.262	0.156	2.15	253.6	68.02	145	6.015	0.909	15.11	1160.6	88.18
85	7.178	0.182	2.54	289.7	69.66	150	5.891	1.022	17.33	1288.4	89.39
90	7.108	0.212	2.98	330.2	71.19	155	5.764	1.145	19.88	1427.4	90.53
95	7.009	0.245	3.50	373.4	72.87	160	5.679	1.333	23.47	1638.7	91.93

By inspection of above table, it will be seen why it is more economical to dry at the higher temperatures. The atmosphere is seldom saturated with moisture, and in practice it will be found generally necessary to heat the air about 30° above the temperature of saturation. Drying on a large scale is now accomplished almost entirely by the "hot-blast" system of heating, details of which may be obtained from the American Blower Company, Buffalo Forge Company, or the B. F. Sturtevant Company.

COMPARISON OF THERMOMETERS.

To convert the degrees of different thermometers from one into the other, use the following formulas:—

F stands for degrees of Fahrenheit, or 212° }
C " " " " Celsius,* " 100° } boiling-point.
R " " " " Réaumur, " 80° }

$F=\frac{9R}{4}+32$, and $F=\frac{9C}{5}+32$ for degrees above freezing-point.

$F=\frac{9R}{4}-32$, and $F=\frac{9C}{5}-32$ for degrees below freezing-point.

* Often called Centigrade.

$C=\frac{5(F-32)}{9}$, and $R=\frac{4(F-32)}{9}$ for degrees above freezing-point.

$C=\frac{5(32-F)}{9}$, and $R=\frac{4(32-F)}{9}$ for degrees below freezing-point.

Degrees Fahr. below zero should be given the $-$ sign.

Zero of Celsius or Réaumur $=+32°$ Fahrenheit. Zero of Fahrenheit $=-17.77°$ C, or $-14.22°$ R.

Ex. 1. How much is 8° Celsius above zero, in Fahrenheit?

$$F=\frac{9\times 8}{5}+32=14.4+32=46.4° \text{ above.}$$

Ex. 2. How much is 8° Celsius below zero, in Fahrenheit?

$$F=\frac{9\times 8}{5}-32=14.4-32=17.6° \text{ above.}$$

IN CASES WHERE THE PRODUCT IS SMALLER THAN 32, IT INDICATES THAT THE DEGREE IS ABOVE ZERO OF FAHRENHEIT; SEE EXAMPLE 2.

Ex. 3. How much is 19° Celsius below zero, in Fahrenheit?

$$F=\frac{9\times 19}{5}-32=34.2-32=2.2 \text{ below Fahrenheit.}$$

DIFFERENT COLORS OF IRON CAUSED BY HEAT.

[Pouillet.]

C.	Fahr.	Color.
210°	410°	Pale yellow
221	430	Dull yellow
256	493	Crimson
261 370	502 680	Violet, purple, and dull blue; between 261° and 370° C. it passes to bright blue, to sea-green, and then disappears
500	932	Commences to be covered with a light coating of oxide, loses a good deal of its hardness, becomes a good deal more impressible to the hammer, and can be twisted with ease
525	977	Becomes nascent red
700	1292	Sombre red
800	1472	Nascent cherry
900	1657	Cherry
1000	1832	Bright cherry
1100	2012	Dull orange
1200	2192	Bright orange
1300	2372	White
1400	2552	Brilliant white, welding heat
1500 1600	2732 2912	Dazzling white

LINEAL EXPANSION OF SOLIDS AT ORDINARY TEMPERATURES.

(British Board of Trade; from Clark.)

	For 1° Fahr.	For 1° Cent.	Coef. of Expansion from 32° to 212° F.	According to Other Authorities.
	Length = 1	Length = 1		
Aluminium (cast)	.00001234	.00002221	.002221	
Antimony (cryst.)	.00000627	.00001129	.001129	.001083
Brass, cast	.00000957	.00001722	.001722	.001868
" plate	.00001052	.00001894	.001894	
Brick	.00000306	.00000550	.000550	
Bronze (Cu, 17; Sn, 2½; Z, 1)	.00000986	.00001774	.001774	
Bismuth	.00000975	.00001755	.001755	.001392
Cement, Portland (mixed), pure	.00000594	.00001070	.001070	
Concerte: cement, mortar, and pebbles	.00000795	.00001430	.001430	
Copper	.00000887	.00001596	.001596	.001718
Ebonite	.00004278	.00007700	.007700	
Glass, English flint	.00000451	.00000812	.000812	
" thermometer	.00000499	.00000897	.000897	
" hard	.00000397	.00000714	.000714	
Granite, gray, dry	.00000438	.00000789	.000789	
" red, dry	.00000498	.00000897	.000897	
Gold, pure	.00000786	.00001415	.001415	
Iridium, pure	.00000356	.00000641	.000641	
Iron, wrought	.00000648	.00001166	.001166	.001235
" cast	.00000556	.00001001	.001001	.001110
Lead	.00001571	.00002828	.002828	
Magnesium	....	....	....	.002694
Marbles, various from	.00000308	.00000554	.000554	
Marbles, various to	.00000786	.00001415	.001415	
Masonry, brick from	.00000256	.00000460	.000460	
Masonry, brick to	.00000494	.00000890	.000890	
Mercury (cubic expansion)	.00009984	.00017971	.017971	.018018
Nickel	.00000695	.00001251	.001251	.001279
Pewter	.00001129	.00002033	.002033	
Plaster, white	.00000922	.00001660	.001660	
Platinum	.00000479	.00000863	.000863	
Platinum, 85 per cent. Iridium, 15 " "	.00000453	.00000815	.000815	.000884
Porcelain	.00000200	.00000360	.000360	
Quartz, parallel to major axis, t 0° to 40° C	.00000434	.00000781	.000781	
Quartz, perpendicular to major axis, t 0° to 40° C	.00000788	.00001419	.001419	
Silver, pure	.00001079	.00001943	.001943	.001908
Slate	.00000577	.00001038	.001038	
Steel, cast	.00000636	.00001144	.001144	.001079
" tempered	.00000689	.00001240	.001240	
Stone (sandstone), dry	.00000652	.00001174	.001174	
" " Rauville	.00000417	.00000750	.000750	
Tin	.00001163	.00002094	.002094	.001938
Wedgewood ware	.00000489	.00000881	.000881	
Wood, pine	.00000276	.00000496	.000496	
Zinc	.00001407	.00002532	.002532	.002942
Zinc, 8 Tin, 1	.00001496	.00002692	.002692	

Note.—Cubical expansion, or expansion of volume = linear expansion × 3.

Systems of Heating.*

The various systems employed for the warming of buildings, aside from the use of stoves and fireplaces, may be classified as follows:

Furnace heating	*Gravity system.* *Fan system.*
Steam heating	*Gravity, or lowpressure systems.* *a.* Direct radiation. *b.* Direct-indirect radiation. *c.* Indirect radiation. *d.* Paul system. *Non-gravity, or high-pressure systems.* *a.* Gravity circulation with return trap or pump. *b.* Webster system. *c.* Hot-blast systems.
Hot-water heating. ..	*Open system.* Direct or indirect radiation. *Closed system.* Direct or indirect radiation.

These systems are briefly described in the following pages and sufficient data given to enable an architect to specify or design, in a general way, an ordinary heating plant. The limits of the book preclude the going into many of the minor details, which are usually left to the judgment of the contractor, or to a discussion of the high-pressure systems, which are generally employed only for very large buildings or for power plants, and for which the plant should be designed by an expert.

For further information on this subject the reader is referred to "Heating and Ventilating Buildings," by Prof. R. C. Carpenter, which for architects and students is the best work on the subject that the author has seen.

Gravity Systems of Steam Heating.

A steam-heating plant may be divided into three distinct parts: 1st, the boiler, or steam generator; 2d, the radiators; and 3d, the supply and return pipes connecting the two.

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* In the preparation of the article on Heating the author has had the assistance o Mr. P. F. Monaghan, an experienced heating engineer.

Radiators.—Radiators are generally made of iron, and may be of any shape that will allow of a good circulation of steam through them, and also permit the air to circulate freely about the outside. It is also desirable that the thickness of the metal shall be only enough to give sufficient strength.

Classes of Radiators.—Radiators are divided into three classes: those affording, 1st, direct radiation; 2d, direct-indirect radiation; 3d, indirect radiation.

Direct radiating surfaces embrace all heaters placed within a room or hall to warm the air already in the room.

Indirect radiating surfaces embrace heating surfaces placed outside the rooms to be heated, and should only be used in connection with some system of ventilation.

Direct-indirect radiation is a mean between the other two methods. The radiators are placed in the rooms to be heated, as in the first method, and a supply of fresh air brought to them through openings in the outside wall of the room or through a space under the lower sash of a window.

Efficiency of Radiators.—The condensation of one pound of steam at 0, or pressure of one atmosphere to water at 212°, gives out 965 thermal units. Hence to determine the amount of heat given out by any radiator in a given time, it is only necessary to determine the amount of water in pounds which the radiator condenses in the same time and multiply it by 965.

The radiator which, under the same conditions of steam pressure and volume and temperature of surrounding air, will condense the most water in a given time is the most efficient.

Measurement of Radiators.—Radiators are rated, or measured, not according to their size, but according to the amount of heating surface coming in contact with the air. The size of radiator for a given amount of heating surface will depend entirely upon the form or shape of the radiator.

Heating by Direct Radiation.—Direct radiation being much more economical than indirect radiation, it will always be much more commonly used for steam or hot-water heating; and in buildings not requiring a great amount of ventilation it offers a nearly perfect mode of heating.

Description of Direct Radiators. *Pipe Radiators.*—The cheapest direct radiator is one formed of wrought-iron pipes (1-inch pipes being generally preferred) placed against a wall one above the other and connected with return bends or branch

tees and elbows, to afford a circulation. The length of pipe required to make up a given amount of heating surface can easily be determined by the use of the table on p. 1205. For rooms in which it is desirable that the heating apparatus shall present a neat appearance and occupy as little space as possible some form of upright radiator is generally employed. Fig. 1 shows a style of radiator, known as a pipe radiator, which was formerly largely used on account of its cheapness; it is now seldom seen, however. Pipe radiators are formed of a number of short upright 1-inch tubes from 2 ft. 8 ins. to 2 ft. 10 ins. long, screwed into a hollow cast-iron base or box, and are either connected together in pairs by return bends at their upper ends or else each tube stands singly, with its upper end closed, and having a hoop-iron partition extending up inside it from the bottom to nearly the top. The radiators are also made circular in form, either in one piece, or in halves for encircling iron columns.

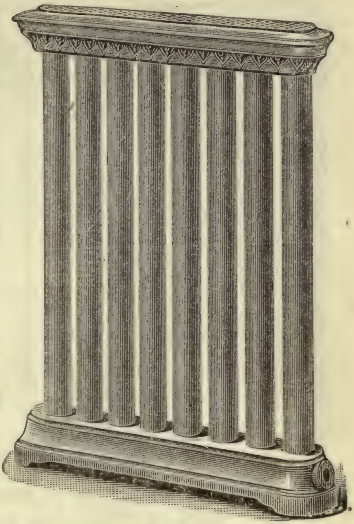


Fig. 1
Direct Pipe Radiator.

The table on next page shows the dimensions of 1-inch pipe radiators for different heating surfaces.

Cast-iron Direct Radiators.—Direct radiators are now made almost exclusively of cast iron.* Within the last decade considerable improvement has been made in the design and quality of cast radiators, so that the newer patterns have very largely superseded those made previous to ten or twelve years ago.

The principal manufacturers of radiators are the "American,"

* Quite recently the Kinnear-Hood Steel Co. has placed on the market a line of sheet-steel, brass, and copper radiators for direct and direct-indirect radiation. The manufacturers claim that they are superior to cast radiators.

TABLE OF VERTICAL PIPE RADIATORS.

No. of Rows and Width of Base.	Tubes in Each Row.	Surface, in Sq. Ft.*	Length. Ft.	Length. In.	No. of Rows and Width of Base.	Tubes in Each Row.	Surface, in Sq. Ft.*	Length. Ft.	Length. In.
Single Rows. Width of Base, 4¼ ins.	4	4	0	10¼	Two Rows. Width of Base, 6¼ ins.	8	16	1	6¼
	6	6	1	2¼		10	20	1	10¼
	8	8	1	6¼		12	24	2	2¼
	10	10	1	10¼		14	28	2	6¼
	12	12	2	2¼		16	32	2	10¼
	16	16	2	10¼		18	36	3	2¼
	20	20	3	6¼		20	40	3	6¼
	24	24	4	2¼		24	48	4	2¼
	28	28	4	10¼		28	56	4	10¼
	32	32	5	6¼		32	64	5	6¼
	38	38	6	6¼		38	76	6	6¼
Three Rows. Width of Base, 8¼ ins.	8	24	1	6¼	Four Rows. Width of Base. 10 ins.	4	16	0	10¼
	12	36	2	2¼		8	32	1	6¼
	16	48	2	10¼		12	48	2	2¼
	20	60	3	6¼		16	64	2	10¼
	24	72	4	2¼		20	80	3	6¼
	28	84	4	10¼		24	96	4	2¼
	32	96	5	6¼		28	112	4	10¼
	38	114	6	6¼		32	128	5	6¼

* For radiators 35 inches high.

the "National," the "United States," the "Penn," and the "Holland" Radiator Companies, the A. A. Griffing Iron Co., the J. L. Mott Iron Works, and the H. B. Smith Co., all of whom make several complete lines. There are also a number of smaller companies who make two or three styles.

The radiators made by the American Radiator Co., however, are probably more extensively used than those of any other make, particularly in the Western States, and it is for this reason that they have been selected for illustration. Nearly all of the patterns made by this company, however, are very closely duplicated by the companies above named, the variation being principally in the ornamentation.

There are some types of radiators which are made for the purpose of circulating steam and hot water in one construction, but the lines of goods made by the American Radiator Co. for water and steam circulation are each made for its own specific purpose.

Figs. 2, 3, and 4 illustrate three of the most popular styles of radiators made by this company, although a large variety of radiators in one-, two-, three-, and four-column and in extended single-column and flue construction are also made by them.

The National and Verona are "two-column radiators"; the Rococo is a "three-column radiator."

Fig. 2
Rococo Radiator.

Fig. 3
National Two-column Radiator.

Fig. 4
Verona Radiator.

Fig. 5 shows three sections of the Colonial wall radiator made by this company, which is very convenient for use in

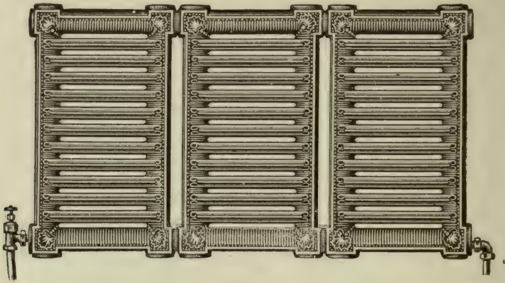


Fig. 5
Three Sections of Colonial Wall Radiator.

halls and bathrooms, as it projects only from 3½ to 4½ ins. from the wall.

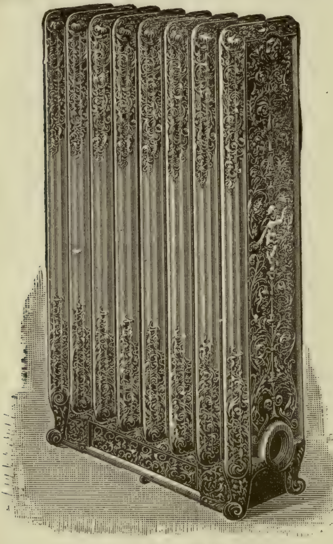


Fig. 6
Italian Flue Box-base Direct-indirect Radiator. Made by Amer. Rad. Co.

This radiator is made in three sizes of sections, 29, 23, and 16¾ ins. long, by 13¼ ins. wide and 2⅝ ins. thick, and containing 9, 7, and 5 sq. ft. of heating surface respectively. The sections may be assembled either horizontally or vertically.

Corner, circular, curved, and column radiators; also dining-room, window, stairway, box-base, and direct-indirect radiators; also such auxiliaries as brackets, pedestals, tops, dampers, and wall-boxes; also special radiator sections with high, low, or single legs are also made by the American Radiator Co., and by all of the other companies above mentioned.

HEATING SURFACE IN SQUARE FEET PER SECTION OF SEVERAL STEAM- AND HOT-WATER RADIATORS MADE BY THE AMERICAN RADIATOR CO.

Name of Radiator.	Length per Section.	Height of Radiator in Inches.					
		45	38	32	26	23	20
National	2½	5	4	3½	2⅔	2½	2
Ideal	2½	"	"	"	"	"	"
Peerless	2½	"	"	"	"	"	"
Perfection	2½	"	"	"	"	"	"
Verona	2½	*	"	"	"	*	"
Italian flue	3	*	7	5¾	4½	*	3¼
		44	38	32	26	22	18
Rococo (ornamental or plain)	2½	6	5	4½	3¾	3	2¼
St. Louis standard, or Buffalo standard, 4 columns	2½	9	8	6⅔	5⅓	4	3
		20	18	16	14	13	
Ætna flue	3	6	5⅓	4⅔	4	3⅔	
Zenith flue	3	6	5⅓	4⅔	4	*	

* Not made in this height.

The width of base of the National, Ideal, and Peerless radiators is 8½ ins.; of the Perfection, 9¼ ins.; Rococo, 10¼ ins.; Buffalo, 12 ins.; St. Louis standard 4 col. and zenith flue, 12¾ ins.; and of the Ætna flue, 12½ ins.

To find number of sections required, divide required heating surface in feet by values given in above table.

To find length of radiator, multiply the number of sections by length per section and add 1 inch for two bushings.

Radiators are generally put together at the factory as ordered. The standard height, except for window radiators, is 38 ins. Heights less than 38 ins. cost a little more.

Direct-indirect Radiation.

The only difference between this method of heating and the direct method is that external air is introduced into the room in such a way that it shall come in contact with the radiator and, becoming heated, circulate through the room, and unless other means are provided pass out through the cracks around the doors and windows. By this arrangement sufficient ventilation is afforded for living-rooms and offices. With direct radiation

no ventilation at all is afforded. There are several methods of arranging the radiators and cold-air inlets, although nearly all require that the radiator shall be located against an outside wall.

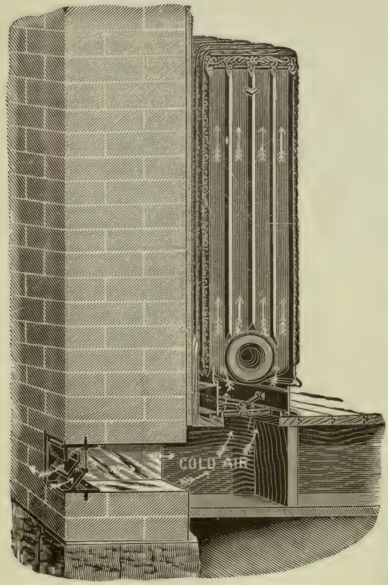


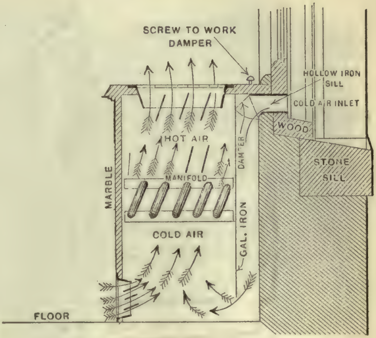
Fig. 7

The simplest method of providing direct-indirect radiation is by using a radiator that has the lower portion encased so as to form a box, as shown in Fig. 6. Cold air can be conducted from the outside of the house through a galvanized iron pipe and admitted to the bottom of the radiator, as in Fig. 7. It is then obliged to pass upward between the radiator flues their entire length and is brought into the room at an exceptionally high temperature. A small damper door is placed in the front of the box, and a damper should also be put in the cold-air supply, so that the radiator can be converted into the ordinary direct type by simply closing the damper and opening the doors. This would probably be required in very cold weather. The outside of the radiator, of course, heats by direct radiation at all times. If a large amount of ventilation is required, some form of indirect radiator should be enclosed in an incombustible casing and the outside air admitted below the radiator. A very good arrangement to accomplish this purpose is shown in Fig. 8.

It consists of a stack of pin or other indirect radiators enclosed in a box of either iron, marble, or wood lined with tin and provided with registers at the top for the escape of the heated air. The cold air enters through a hollow iron sill placed above the wooden-sill of a window and passes down back of the

radiator, through a galvanized iron pipe, to the space under the radiator.

The cold-air inlet is provided with a damper so that it can be closed, and registers are also placed at the base of the radiator casing, so that in very cold weather the cold-air inlet may be partially or wholly closed and the air allowed to circulate through the bottom register, up through the radiator, and out of the top registers.



VERTICAL SECTION THROUGH RADIATOR, CASING AND WINDOW SILL.

Fig. 8

Indirect Radiation.

Heating by indirect radiation is accomplished by two methods, the more general method being to have separate radiators for each room, located in the cellar or basement, incased with metal or wood lined with tin and provided with a fresh-air inlet and tin pipe to convey the hot air to the room to be heated.

The other method is to provide one cold-air inlet for the whole building and place a large coil of steam-pipes behind it, so that all the air entering the building must pass through this coil. Such a method can only be used in connection with fan ventilation.

Fig. 9 shows the usual method of casing indirect radiators. The casing is generally of galvanized iron or of wood lined with tin. The latter is best when the cellar is to be kept cool, as there is a greater loss by radiation and conduction through metal cases; otherwise metal is best, as it will not crack, and when put together with small bolts can be removed to make repairs without damage.

The boxes should be fitted with a door in the bottom, and the cold-air pipe should always be provided with a damper.

The vertical air-ducts are usually tin flues built into the wall when the building is going up. Sometimes they are only plas-

tered; but round, smooth metal linings with close joints give much the best results. The cross-section of the air-duct should be comparatively large, as a large volume of warmed air with a slow velocity gives the best results.

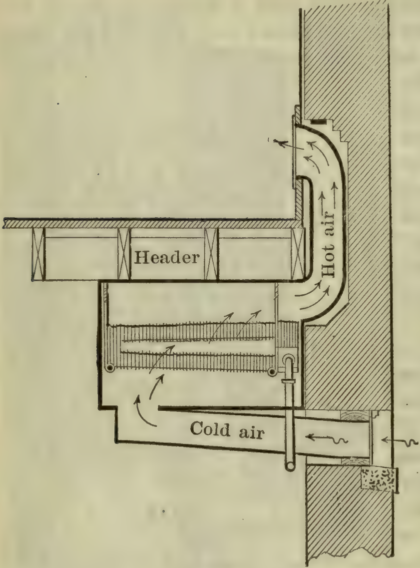


Fig. 9
Section through Indirect Radiator Stack.

There should be a separate vertical air-duct for every outlet or register. In branched vertical air-ducts one is generally a failure. The heated air from one heater may be taken to two or more vertical air-ducts when they start directly over it; the duct to the lower room being taken from the top and that to the upper room from the side, or both from the top. If both rooms are on the same level, both ducts should be taken from the top of the box.

Inlet or cold-air ducts are best when there is one for every coil or heater. Sometimes only one large-branched cold-air duct is used, but this system will give trouble unless all the rooms are ventilated by forced ventilation.

The Radiators.—For indirect radiation a form of radiator is employed different from those used for direct heating. In this method the desideratum is to have as many feet of heating surface in as little space as possible, appearance being of no importance. The earliest form used, and which is still used in the fan or hot-blast systems, is the pipe-coil radiator, formed of a coil of pipes connected at the ends with return bends.

For ordinary indirect heating cast-iron radiators of one of the types shown by Figs. 10, 11, and 12* are now used almost

* From the catalogue of the American Radiator Co.

exclusively, as they are fully as cheap if not cheaper than pipe radiators and more satisfactory.

The pin radiator is made by several manufacturers and is one of the earliest types of indirect radiators.

The radiator shown by Fig. 10 is made in two types: for

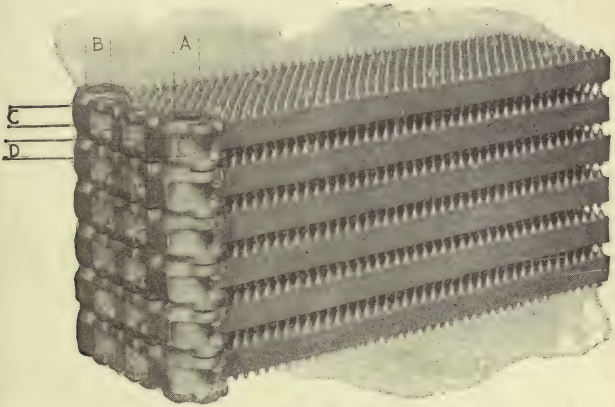


Fig. 10
Perfection Pin, Extra Large, Flange and Bolt.

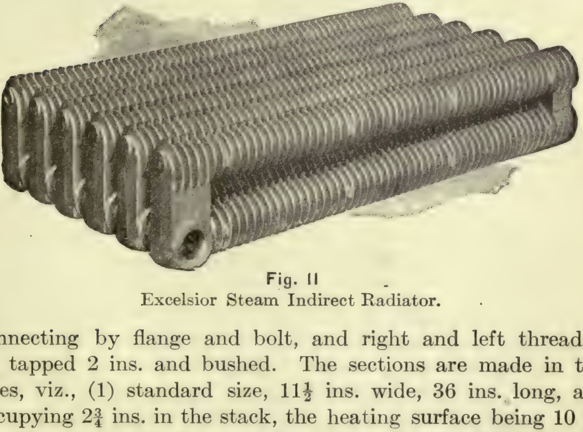


Fig. 11
Excelsior Steam Indirect Radiator.

connecting by flange and bolt, and right and left threaded, all tapped 2 ins. and bushed. The sections are made in two sizes, viz., (1) standard size, $11\frac{1}{2}$ ins. wide, 36 ins. long, and occupying $2\frac{3}{4}$ ins. in the stack, the heating surface being 10 sq. ft. per section; (2) the extra large size, which is $15\frac{1}{2}$ ins. wide, 36 ins. long, occupies $2\frac{7}{8}$ ins. in the stack and has a heating

surface of 15 sq. ft. per section. The Excelsior pattern, shown by Fig. 11, is $36\frac{3}{4}$ ins. long, 8 ins. wide, occupies $3\frac{3}{8}$ ins. in the stack, has a heating surface of 12 sq. ft. per section, and is tapped $1\frac{1}{2}$ ins. The Sterling, Fig. 12, is 37 ins. long, 16 ins. high, occupies $3\frac{1}{2}$ ins. in the stack, contains 20 sq. ft. per section, and is tapped 2 ins. and bushed.

Cast-iron indirect radiators with *plain* surfaces are also made by the "American" and by some of the other radiator companies.

Nearly all indirect radiators may be used for either water or

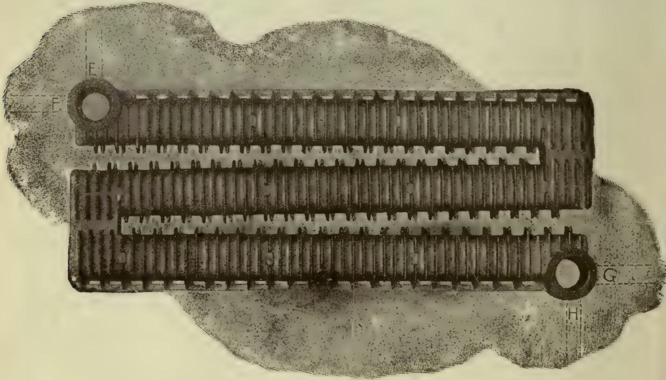


Fig. 12
Sterling Indirect Radiator.

steam circulation, although the American Radiator Company has slightly different patterns for steam than for water.

Indirect radiators are generally hung from the ceiling by four iron hangers attached to the floor joists and having their lower ends shaped so as to hold iron pipe or bar iron on which the radiator rests. The front support should be $\frac{1}{4}$ in. lower than the rear, so that the upper pipe of each radiator will incline to the rear and the lower pipe of each will incline to the front. By this arrangement the water of condensation will follow the course of steam throughout each section. The outlet side of each stack should be from $\frac{1}{2}$ to $\frac{3}{4}$ of an inch lower than the inlet side so as to allow the water free passage through and out of the stack.

Each stack of radiators shouid have, in the warm-air cham-

ber, not less than 12 ins. clear space above them and not less than 6 ins. below them. The *supply* and *return* pipes should always be of ample size.

The space required for any quantity of heating surface of any one of the three radiators described above may be readily determined by means of the data given. The following table will be found useful in proportioning size of air-ducts:

DATA FOR EXCELSIOR INDIRECT STEAM-RADIATORS.

Heating Surface.	Cold-air Supply.	Diameter of Duct if Round.	Hot-air Flue.	Size for Brick-work if Hot-air Flues.	Size of Register.	Ratio of 1 to 30.	Ratio of 1 to 35.	Ratio of 1 to 40.
Sq.Ft.	Sq.In.	Inches.	Sq. In.	Inches.	Inches.	Cu. Ft.	Cu. Ft.	Cu. Ft.
24	36	6.8	48	4×12	8×8	720	840	960
36	54	8.3	72	8×12	9×12	1080	1260	1440
48	72	9.6	96	8×12	10×14	1440	1680	1920
60	90	10.0	120	12×12	12×15	1800	2100	2400
72	108	11.7	144	12×12	12×19	2160	2520	2880
84	126	12.7	168	12×16	14×22	2520	2940	3360
96	144	13.5	192	12×16	14×24	2880	3360	3840
108	162	14.4	226	12×20	16×20	3240	3780	4320
120	180	15.2	240	12×20	16×24	3600	4200	4800
132	198	15.9	264	12×24	20×20	3960	4620	5280
144	216	16.6	288	12×24	20×24	4320	5040	5760

The Boiler.

Classes of Heating Boilers.—There are a great many varieties of steam-boilers in use for generating steam for heating purposes besides several types that were on the market some twelve or fifteen years ago and are now practically obsolete. The larger proportion of the boilers used at the present time may be classed under the following heads, viz.:

(1) Horizontal tubular boilers.

(2) Fire-box boilers.

(3) Sectional boilers.

a. Boilers with vertical sections.

b. Boilers with horizontal sections.

Horizontal Tubular Boilers.—This boiler has been very extensively used both for heating and power and is still

preferred by many engineers for heating large buildings or generating steam for hot-blast heating systems. It is an efficient type of boiler, is easily cleaned, and is usually the most economical type for a large amount of radiation, say over 2,500 sq. ft., and particularly when soft coal is used for fuel.

The chief objection to its use is that should an explosion occur from any cause, it is liable to do a great amount of damage, possibly demolish the building. The chance of an explosion, however, is very small indeed.*

Tubular boilers are manufactured in nearly every city of importance and can be purchased in every market at a reasonable price.

The boiler should be provided with manholes with strongly reinforced edges, so that a person can enter for cleaning. The heads of the boiler above the tubes should be thoroughly braced in order to sustain safely any pressure from the inside of the boiler.

Domes.—Domes are often placed above the horizontal part of a boiler for the purpose of increasing the capacity for the storage of steam and to afford a ready means of drawing off dry steam.

The desirability of domes is a much disputed question. The dome is always an element of weakness in a boiler, and many engineers claim that the boiler is better without them. For gravity heating, boilers without domes are probably most used, while for power purposes the dome is generally provided. There seems to be no *standard proportions* for tubular boilers, as the practice of different makers and engineers varies somewhat. The proportions given in the following tables, however, are fairly representative of most of the boilers made, those in

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* "The claim for safety can also be made for the horizontal return tubular boiler. The fact is that when boilers of this type are properly constructed they do not explode. When one compares the few explosions which occur with the great number of boilers of this type which are manufactured every year and the vastly greater number which are in use, a large number of them carelessly constructed and carrying a greater pressure than they were designed to carry, it is a strong argument in support of this claim, that they are safety boilers when proper care and inspection are given to their construction.

"Moreover, the horizontal return tubular boiler when well designed and carefully constructed is not only a safety boiler, but when compared with water-tube boilers we do not hesitate to say that it is more economical." (Edward Kendall & Sons.)

the first table being designed for hard coal and those in the second table for soft coal.

HORIZONTAL TUBULAR BOILERS.

MANUFACTURED BY EDWARD KENDALL & SONS, CAMBRIDGE, MASS.*

Diameter of Boiler.	Length of Boiler.		Number of Tubes.	Diameter of Tubes.	Length of Tubes.	Thickness of Shell.	Heating Surface.	Nominal Horse-power.	Approx. Weight of Boiler and Castings.	Square Feet of Grate Surface.†	Square Feet of Radiating Surface that Can be Supplied.‡
Ins.	Ft.	Ins.		Ins.	Ft.	Ins.	Sq. Ft.				
30	6	0	36	2	5	1/4	114	7 2/3	3,600	5	684
"	7	0	"	"	6	"	137	9 1/5	3,750	5	822
"	8	0	"	"	7	"	160	10 2/3	3,900	6	990
"	9	0	"	"	8	"	182	12	4,050	8	1,092
36	8	0	34	2 1/2	7	"	189	12 1/2	4,390	8	1,124
"	9	0	"	"	8	"	216	14 1/2	4,600	8	1,296
"	10	0	"	"	9	"	243	16	4,810	10	1,458
"	11	0	"	"	10	"	270	18	5,090	10	1,620
"	12	0	"	"	11	"	297	20	5,300	12	1,782
"	13	0	28	3	12	"	321	21	5,510	12	1,926
42	10	0	45	2 1/2	9	"	315	21	6,610	12	1,890
"	11	0	"	"	10	"	350	23	7,030	12	2,100
"	12	0	"	"	11	"	384	26	7,300	14	2,304
"	13	0	"	"	12	"	420	28	7,660	14	2 520
"	12	0	38	3	11	"	389	26	7,320	14	2,334
"	13	0	"	"	12	"	425	28	7,680	14	2,550
"	14	0	"	"	13	"	460	31	7,950	16	2,760
"	15	0	"	"	14	"	495	33	8,220	16	2,970
48	12	2	69	2 1/2	11	5/16	566	38	9,750	18	3,396
"	13	2	"	"	12	"	617	41	10,150	18	3,702
"	15	2	49	3	14	"	626	42	10,685	18	3,756
"	16	2	"	"	15	"	671	45	11,035	18	4,026
"	17	2	"	"	16	"	716	48	11,485	20	4,296
"	17	2	38	3 1/2	16	"	658	44	12,085	20	3,948
"	18	2	"	"	17	"	700	47	12,535	20	4,200
54	15	2	60	3	14	11/32	759	51	14,015	24	4,554
"	16	2	72	"	15	"	954	63	15,074	26	5,724
"	17	2	"	"	16	"	1,018	68	15,584	28	6,108
"	18	2	"	"	17	"	1,082	72	16,094	28	6,492
"	17	2	54	3 1/2	16	"	905	60	15,458	26	5,430
"	18	2	"	"	17	"	961	64	15,960	26	5,766
"	19	2	"	"	18	"	1,018	68	16,552	28	6,108
60	18	2	92	3	17	3/8	1,364	91	19,000	34	8,284
"	18	2	64	3 1/2	17	"	1,133	76	18,468	30	6,798
"	19	2	"	"	18	"	1,200	80	19,227	32	7,200
66	18	2	110	3	17	"	1,615	108	22,430	40	9,690
"	18	2	82	3 1/2	17	"	1,426	95	22,190	36	8,556
72	18	2	130	3	17	7/16	1,900	127	26,036	48	11,400
"	18	2	100	3 1/2	17	"	1,721	115	25,980	44	10,326

* Selected from 156 sizes listed by this firm. These boilers are made up to 96 ins. diam. and 21 ft. long.

† For hard coal or coke.

‡ Proportion 6 to 1. The last two columns added by the author.

When soft coal is used for fuel the efficiency of the boiler may be increased by increasing the grate area about 20 per cent.

PROPORTIONS OF HORIZONTAL TUBULAR BOILERS.

Made by the Atlas Engineering Works, Indianapolis, Ind.
These are about the standard proportions as used in the Western States for ordinary purposes.

Nom. Rated H.-P.*	Shell.† Diameter.	Shell.† Length.	Mean Thickness. Shell.	Mean Thickness. Heads.	Tubes. Number.	Tubes. Diameter.	Tubes. Length.	Heating Surface.	Grate Surface.
	Inches	Feet.	Inches.	Inches.		Ins.	Feet.	Sq. Ft.	Sq. Ft.
15	36	8	1/4	3/8	26	3	8	214	5.8
20	36	10	1/4	3/8	26	3	10	266	8.3
25	36	12	1/4	3/8	26	3	12	318	9.5
30	40	12	1/4	3/8	34	3	12	404	12
35	42	12	1/4	7/16	40	3	12	464	12.8
40	46	12	9/32	7/16	42	3	12	491	14.6
45	48	12	9/32	7/16	48	3	12	551	15.3
50	48	14	9/32	7/16	40	3½	14	630	16
55	52	14	5/16	7/16	44	3½	14	693	16.7
60	54	14	5/16	1/2	46	3½	14	721	18
70	54	16	5/16	1/2	40	4	16	817	20.8
75	60	14	11/32	1/2	62	3½	14	940	21.5
85	60	16	11/32	1/2	52	4	16	1045	22.2
100	66	16	3/8	1/2	64	4	16	1265	25
125	72	16	7/16	1/2	82	4	16	1578	29.5
150	72	18	7/16	1/2	82	4	18	1775	36.5

* It will be noticed that these boilers are rated a little higher than the usual standard of 15 sq. ft. of heating surface to 1 H.-P.

† In these boilers the smoke-box is made of a separate piece, so that the actual length of boilers is 15 ins. more than length of shell.

Size of Tubes.—In the Eastern States where hard coal is used, 2½-inch tubes are commonly placed in boilers up to 12 ft. long, but where soft coal is used for fuel, the tubes should not be less than 3 ins. in diameter even for the smallest boiler, while for boilers 16 ft. long and over 3½ ins. or 4 ins. tubes should be used.

Setting of Horizontal Tubular Boilers.—Boilers are set with *half fronts* and *full fronts*. With half-front setting, the front end of the boiler projects 12 ins. or more beyond the brickwork and is covered with a cast-iron frame containing two doors for giving access to the flues.

With a full front, a cast-iron front is provided the full width

of the boiler and extending from the floor to the top of the brick setting.

Fig. 13 shows the proper method of setting a horizontal tubular boiler with full front, and the table * opposite it gives the dimensions indicated by the letters and the quantities of bricks required. These will be found useful in showing the boiler setting on the foundation plan of the building, and also in estimating the cost of setting.

Fire-box Boilers.—A fire-box boiler is a horizontal tubular boiler with a fire-box formed in the front end, as in Fig. 14. The fire-box has double walls, the space between being filled with water, so that the fire is entirely surrounded with water, the object being to utilize a greater percentage of the heat generated by combustion than is possible with the ordinary tubular boiler.

The American Radiator Co. and the Kewanee Boiler Co. make a fire-box boiler intended especially for heating purposes, which would seem to be a very efficient type of boiler for buildings having from 1,000 to 3,000 ft. of direct steam radiation or 1,500 to 6,000 ft. of hot-water radiation, and particularly where hard coal or coke is used for fuel.

These boilers may be installed in very low cellars. The danger from explosion with these boilers, however, when used for steam heating is about the same as with plain tubular boilers.

Fire-box boilers require a brick setting as shown by Fig. 14.

Sectional Cast-iron Boilers.—This class of boiler has been used for a great many years, but during the past ten years they have become more popular than ever, and are now very largely taking the place of tubular boilers for the heating of quite large private and public buildings, principally on account of their safety from dangerous explosions. This, in fact, is the chief advantage of the sectional boiler over a tubular boiler. In a sectional boiler, should an explosion occur from gross carelessness of the attendant, it would probably be confined to not more than two sections, and do but little damage to the building. Many improvements have been made in these boilers during the past decade, so that some of the latest patterns seem to be about perfect for the class of work for which they are intended.

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* Published by Kellog-Mackay-Cameron Co.

MEASUREMENTS FOR SETTING TUBULAR BOILERS WITH FULL FRONTS.

Reference letters on diagram.

A	B	C	E	F	G	H	I	J	K	L	M	N	O	P	Q	R		S		Com. Brick.	Fire-brick.
Ins.	Ft.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ft.	Ins.	Ft.	Ins.		
30	8	12½	14	30	44	80	18	18	18	8	18	13	35	16	36	5	2	11	7½	5,200	320
30	10	12½	14	36	50	80	18	18	18	8	18	13	35	16	36	5	2	13	7½	5,800	320
36	8	12½	14	36	50	80	18	18	18	8	18	13	35	16	42	5	8	11	7½	6,200	480
36	10	12½	14	36	50	80	18	18	18	8	18	13	35	16	42	5	8	13	7½	7,000	480
40	10	14¾	14	30	44	92	21	20	21	8	18	18	40	21	46	6	10	14	2¾	7,700	600
40	12	14¾	14	36	50	92	21	20	21	8	18	18	40	21	46	6	10	16	2¾	8,800	600
42	10	14¾	14	36	50	92	21	20	21	9	20	18	40	21	48	7	0	14	4¾	10,000	720
42	12	14¾	14	42	56	92	21	20	21	9	20	18	40	21	48	7	0	16	4¾	10,800	720
42	14	14¾	14	42	56	92	21	20	21	9	20	18	40	21	48	7	0	18	4¾	11,600	720
48	12	14¾	15¾	42	57¾	100	22	21	24	9	20	18	42	21	54	7	6	16	4¾	13,200	980
48	14	14¾	15¾	48	63¾	100	22	21	24	9	20	18	42	21	54	7	6	18	4¾	14,200	980
48	16	14¾	15¾	54	69¾	100	22	21	24	9	20	18	42	21	54	7	6	20	4¾	15,200	980
54	14	16½	15¾	48	63¾	110	25	22½	24	10	24	22	46½	25	60	8	8	19	2½	14.900	1 154
54	16	16½	15¾	54	69¾	110	25	22½	24	10	24	22	46½	25	60	8	8	21	2½	16,000	1 154
60	14	16½	16	54	70	118	27½	22½	24	10	24	22	49	25	66	9	2	19	2½	16,100	1,280
60	16	16½	16	54	70	118	27½	22½	24	10	24	22	49	25	66	9	2	21	2½	17,400	1 280
60	18	16½	16	60	76	118	27½	22½	24	10	24	22	49	25	66	9	2	23	2½	18,700	1,280

Types of Sectional Boilers.—There is such a great variety of small sectional boilers for house heating that it is

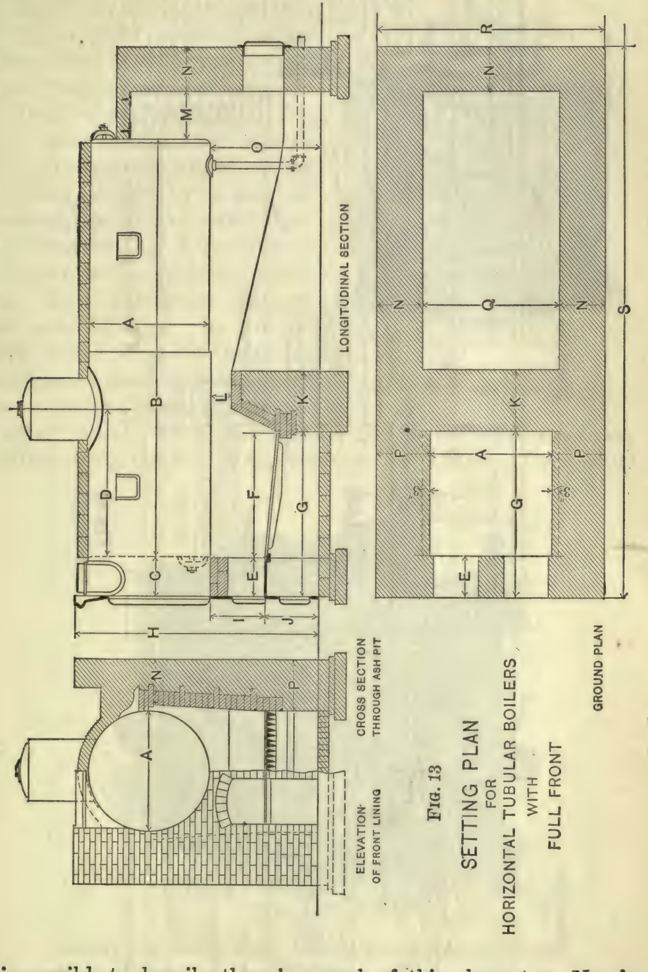

FIG. 13

SETTING PLAN FOR HORIZONTAL TUBULAR BOILERS WITH FULL FRONT

impossible to describe them in a work of this character. Nearly all are of a portable pattern and are generally made in horizontal sections, i.e., the sections fitting one on top of the other.

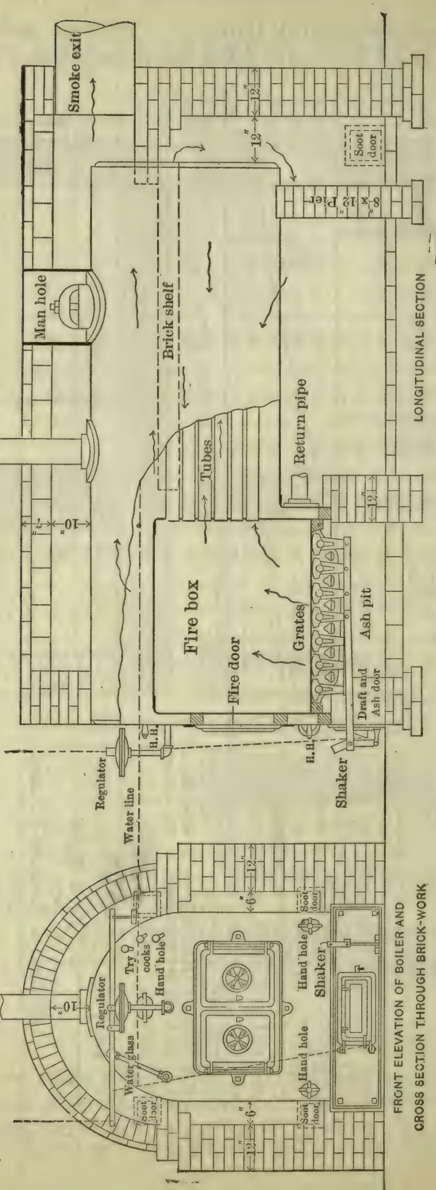

Fig. 14
Fire-box Boiler.

For buildings having more than 400 ft. of direct radiation—in the radiators—a vertical sectional boiler is to be preferred to one with horizontal sections.

The vertical sectional boiler is made up of a number of cast-iron vertical sections set one in front of the other on a cast-iron base which forms the ash-pit. The sections are connected together either by means of push nipples fitting tightly into adjacent sections or by connecting each section to three drums, one above the boiler and one on each side near the bottom. The latter type of boiler is designated as a "screw-nipple" boiler and the former as a "push-nipple" boiler. The push-nipple boiler is the later type and seems to be most in favor with steam-fitters. It affords a freer circulation of steam and water than the screw-nipple type and is more quickly erected.

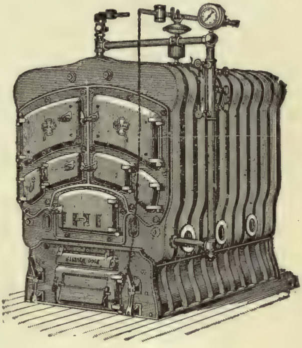

Fig. 15
"Ideal" Sectional Steam Boiler.

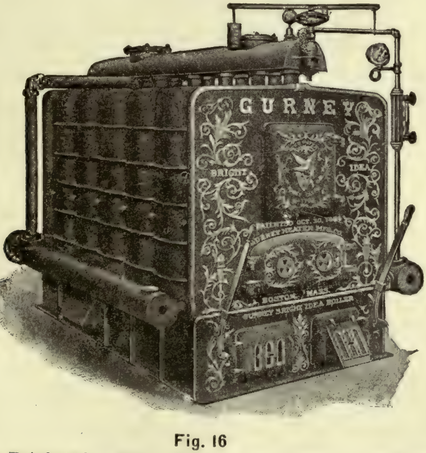

Fig. 16
Gurney Bright Idea Boiler, 1200 Series—Screw-nipple Type.

On the other hand, if any section of a push-nipple boiler

becomes disabled, the entire boiler must be thrown out of use until a new section can be put in, while with the screw-nipple boiler, if an intermediate section is disabled it can be disconnected from the drums and the openings plugged, so that the boiler can be run temporarily, or even permanently, if the boiler is large enough, without taking out the disabled section.

Fig. 15 shows the general external appearance of push-nipple boilers and Fig. 16 of the screw-nipple boilers. Fig 17 is a sectional view of the Gurney Bright Idea sectional water-tube hot-water boiler, "1200 series," adapted to from 5,000 to 11,000 ft. of radiation. Boilers constructed on the same principle are also made for house heating.

Although the external appearance of nearly all vertical sectional boilers is quite similar, the arrangement of the flues or

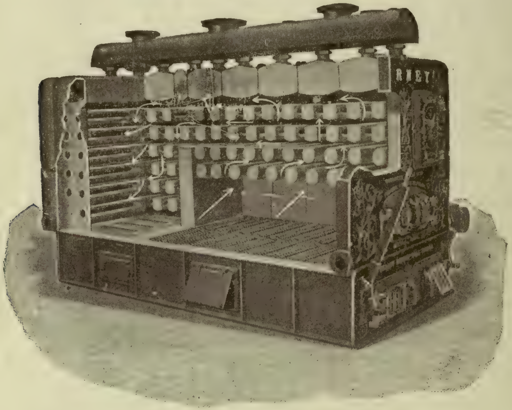

Fig. 17
Sectional View of Gurney Boiler.

passages for the gases of combustion differs somewhat in each different line of boilers, and between some lines, as between the Gurney "Bright Idea" and the "Ideal" line of the American Radiator Co., is very great. In general, it may be said that those heaters which have the greatest amount of heating surface in proportion to the grate area are likely to prove the most efficient in point of economy of coal consumption.

As a rule, the intermediate sections of sectional boilers are alike, so that the capacity of the boiler may be varied within certain limits by increasing the number of sections.

The requirements of an economical and satisfactory working boiler for house heating are as follows:

First.—They should contain a quantity of water sufficiently large to fill the pipes and radiators with steam to any required pressure *without lowering the water in the boiler to require an addition when steam is up*, for should the steam go down suddenly, there will be too much water in the boiler This occurs in boilers made with very small parts or pipes which have a small capacity at the water-line; such boilers require great care for should the boiler have an automatic water-feeder set for the *true* water-line, it will fill up, but cannot discharge again when the steam goes down, while if it has no feeder, there is danger of spoiling the boiler, as the water is in the pipes *in the form of steam.*

It is true that a boiler which contains a small amount of water in proportion to its heating surface will *get up steam* quicker than one containing a larger quantity of water, but the latter will keep steam much better when the fire is renewed; and boilers which contain small quantities of water are rapidly chilled as well as rapidly heated and must be fired often and regularly.

Second.—The fire-box should be of iron, with a water space around it, to prevent clinkering on the sides and the necessity of repairs to brickwork, which are unavoidable in brick furnaces.

Third.—The fire-box should be *deep* below the fire-door, to admit of a thick fire to last all night and thus keep up steam. For large boilers which require the services of an engineer it is desirable to have a large grate area and a thin fire; but such a fire requires to be renewed too often to be suitable for a house boiler.

Fourth.—The fire-box should be *spacious*, for the sake of good combustion.

Fifth.—The boiler should have few parts, and the *flues and tubes should be large* and in a vertical position, so that they will not foul easily, and that any deposit may fall to the bottom.

For dwellings the writer advises those forms of boilers which are without tubes, or with but a very few, as the tubes will invariably give out long before the shell, and if the tubes are not kept clean they will transmit but a small percentage of heat.

Sixth.—All parts should be *readily accessible for cleaning and repairs.* This is a point of the greatest importance and economy. When the heating surfaces become covered with soot and ashes,

the economy of the boiler greatly decreases, as the soot acts as an insulator and prevents the heat reaching the boiler. It is for this reason that boilers which work well when new are found insufficient to do the work required of them when they become dirty.

Seventh.—The heating surface should be arranged as nearly as possible at right angles to the currents of heated gases and so break up the currents as to extract the entire available heat therefrom.

Eighth.—It should have, if possible, *no joints exposed to the direct action* of the *fire*.

Ninth.—It should have a great excess of strength over any legitimate strain, and should be so constructed as not to be liable to be strained by unequal expansion.

Tenth.—It should be durable in construction and not liable to require early repairs.

Eleventh.—The water space should be divided into sections, so arranged that should any section give out no general explosion can occur and the destructive effects be confined to the simple escape of the contents.

Twelfth.—It should be proportioned for the work to be done, and be capable of working to its full rated capacity with the highest economy.

Thirteenth.—It should be provided with the very best gauges, safety-valves, and other fixtures.

The boiler should be set so that the water-line in the boiler will be at least 2 ft. below the main horizontal supply-pipe.

The more prominent lines of sectional boilers for low-pressure steam or hot-water heating are: The "Ideal" line, made by the American Radiator Co.; the "International" and "Carton," made by the International Heater Co.; the "Bright Idea," made by the Gurney Heater Mfg. Co.; the "Mercer" and "Gold," made by the H. B. Smith Co.; the "Furman," made by the Herendeen Mfg. Co.; the "Sunray," made by the J. L. Mott Iron Works; the "Burnham," made by Lord & Burnham Co., and the "Florence," made by the Columbia Heating Co.

The "Ideal" boilers are very extensively used throughout the Western States. Besides being a very efficient boiler they also have the merit of being very low in stature, thus fitting them for installation in buildings having very low cellars without the necessity of constructing special pits. The American Radiator Co. makes twenty different types of sectional boilers

for steam and water, adapted to all kinds of fuel, and fifteen different types of round boilers.

Setting and Covering of Sectional Boilers.—The only brickwork required for any of the boilers named above is a suitable foundation with water-tight ash-pit about 12 ins. deep.

The outside of the boilers, however, should be plastered with a substantial covering, from 1 to 1½ ins. thick, of plastic asbestos.

Rating of Steam-boilers.—Tubular boilers are often designated as so many "horse-power."

Strictly speaking, there is no such thing as "horse-power" to a steam-boiler, as it is a measure applicable only to dynamic effect. But as boilers are necessary to drive steam-engines, the same measure applied to steam-engines has come to be universally applied to the boiler and cannot well be discarded.

The standard established by the committee of judges of the Centennial Exposition in 1876, and since adopted by the A. S. M. E., is "the evaporation of 30 lbs. of water per hour from feed-water at 100° Fahr. into steam at 70 lbs. gauge pressure." This standard is equal to 33,305 thermal units per hour. As the amount of water which any boiler will evaporate per hour depends as much upon the management of the fire and the kind of fuel used as upon the size, the above standard is a difficult one to determine with accuracy, so that in practice the commercial horse-power of a boiler has come to be measured by the amount of its heating surface, i.e., the heating surface available in generating steam.

It is the general practice to consider 15 *sq. ft. of heating surface in horizontal tubular boilers and* 11.5 *sq. ft. in water-tube boilers as equivalent to one horse-power*, and most manufacturers rate their boilers by this standard.

The heating surface of horizontal tubular boilers is computed as follows, all dimensions being taken in inches. Multiply two thirds the circumference of the shell by its length, multiply the sum of the circumferences of all the tubes by their common length; to the sum of these products add two thirds of the area of both tube sheets less twice the combined area of all the tubes and divide the sum by 144 to obtain the result in square feet. Or, the heating surface is equal to the surface area of all the tubes plus two thirds the surface of the shell and both tube sheets minus the area of the holes.

Steam-heaters, i.e., boilers intended only for the heating of buildings, are generally rated by the manufacturers according to the amount of direct radiating surface they will supply, including all piping. These ratings are commonly made pretty high, so that it is a safe rule to use a boiler having a rating 40 per cent. in excess of the actual direct radiation (radiators) when the mains are covered and 50 per cent. when they are not covered.

Each foot of indirect radiation should be figured as equal to $1\frac{3}{4}$ ft. of direct radiation.

Proportioning Radiating Surface to Horizontal Tubular Boilers.—To determine the size of boiler necessary to supply a given amount of direct radiation, allow 1 sq. ft. of heating surface in the boiler to 6 to 7 sq. ft. of direct radiation when all mains are covered and 1 to 5 or 6 sq. ft. when the mains are not covered. A large boiler will usually supply a greater amount of radiation in proportion to its heating surface than a small one.

In these rules the piping is not to be included in the radiating surface.

It should be borne in mind that no hard and fast rule can be given for proportioning heating surfaces, hence in laying out a heating plant the architect will do well to be guided to some extent by the advice of an experienced steam-fitter.

Amount of Coal Burned per Hour.—"The amount of coal burned per square foot of grate surface per hour is rarely less than 15 lbs. with power boilers, and in some cases is very much greater, but it is usually less than 10 lbs., and is sometimes as small as 3 or 4 with heating boilers." *

Boiler Trimmings.—Every steam-boiler should be provided with a brass-cased steam-gauge, safety-valve, and water-column with gauge, water-gauge, and glass. An automatic damper regulator with connections for operating draft door and cold-air check is also desirable on house heaters. The best safety-valve for low-pressure boilers is the single weighted type; it should be connected at the top of the heater.

SYSTEMS OF PIPING FOR STEAM HEATING.

Distinction Between Gravity and Non-gravity Systems.—The various systems of steam heating are divided

* Prof. R. C. Carpenter.

into two general classes, viz., *gravity circulating systems* and *non-gravity systems*. The former embraces all systems in which the water of condensation from the various radiators returns to the boiler by its own weight, i.e., by gravity, without the aid of any mechanical device.

Non-gravity systems require some special machinery, such as a pump or return trap, to return the water to the boiler or in some cases the water of condensation is wasted.

The kind of boiler used or the character of the radiation has nothing to do with the distinction between the two systems, although with the non-gravity systems tubular or power boilers are generally employed. Wherever high-pressure steam is carried on the boiler, the non-gravity system must be used, hence this system is often designated as the high-pressure system, but it is very seldom that high-pressure steam is carried into the radiators. If high-pressure steam is generated for power purposes, that portion of live steam which is used for heating is generally passed through a reducing valve, so that the pressure in the radiators does not exceed 10 lbs., and if exhaust steam is used it can be mixed with the reduced live steam; otherwise the heating system is exactly the same as a gravity system, except in returning the water of condensation to the boiler. On the other hand, where low-pressure steam is used and it is necessary to place radiators below the water-line in the boiler, a non-gravity system must be used because the water of condensation must be collected in a tank or receiver and returned to the boiler by a return trap or pump. For gravity circulation the lowest radiation must be at least 4 ft. above the water-line in the boiler.

The same system of piping may be used for both systems, except that with the non-gravity systems the return pipe must terminate in a tank or receiver placed below the level of the lowest radiator.

Definitions of Terms Used in Describing Steam and Hot-water Piping.—There are certain terms used in describing steam or hot-water piping with which an architect or superintendent should be familiar.

The *main* or *distributing* pipe is the pipe leaving the boiler and which conveys the steam or hot water to the risers or branches which supply radiating surfaces. In steam heating this pipe is termed the *main steam-pipe*, and in hot-water heating the *main flow pipe*. The term *supply* pipe is some-

times applied to main steam-pipes, but it is not technically correct.

The pipes in which the flow takes place from the radiator are called *return pipes.* The *main return* is the pipe which connects with the boiler below the water-line, or, in a non-gravity system, connects with the receiver

Risers are those pipes which extend in a vertical direction to supply radiators. The vertical pipes in which the flow is downward are called *return* risers.

A *relief* or *drip* is a small pipe run from a steam-main to a return. It must be used at all points where water is likely to gather in the main.

Pitch is the inclination given to any pipe when running in a nearly horizontal direction.

The term *water-line* is used to denote the height at which the water will stand in the return pipes. In a gravity system the water-line is practically the level of the water in the boiler.

"*Water-hammer* is a term applied to a very severe concussion which often occurs in steam-heating pipes and radiators. It is caused by cold water accumulating to such an extent as to condense some of the steam in the pipe, thus forming a vacuum which is filled by a very violent rush of steam and water. The water strikes the side of the radiators or pipes with great force and often so as to produce considerable damage. In general, water-hammering may be prevented by arranging the piping in size and pitch so that the water of condensation will immediately drain out of the radiators or pipes." *

An *air-trap* is an upward bend in a pipe which accumulates air to such an extent as to prevent circulation in the system. When an air-trap cannot be avoided, a small pipe or air-valve for the escape of air should be connected with the highest portion of the bend and led to some pipe which will freely discharge the entrapped air.

Systems of Steam Piping.—Three systems of piping are employed in gravity steam heating which may be briefly described as follows:

First. The Mills, or Complete-circuit System (introduced into this country by J. H. Mills and sometimes called the "overhead single-pipe system").—In this system the main pipe is led directly to the highest part of the building, usually to the attic,

* Prof. Carpenter.

from whence distributing pipes are run to the various return risers, which extend to the basement and discharge into the main return. The supply for the radiating surfaces is all taken from the return risers, and in some cases the entire downward circulation passes through the radiating system.

In this system the radiators in the top story receive steam first, and the steam and water of condensation is always flowing in the same direction except in the main steam riser. But one connection is made to each radiator, the steam and water of condensation flowing through the same opening and riser. Below the first floor the piping carries only the return water and steam.

"This system is equally well adapted for either steam or hot-water heating, and on the score of positiveness of circulation and ease of construction is no doubt to be commended as superior to all others." * It is also the best system for compensating for the expansion in the risers in tall buildings. The principal objections to it are (1) the horizontal distribution pipes having to be in the attic or top story instead of in the basement, which may or may not be of serious importance; and (2) the cost of piping is a little greater than with the usual one-pipe system, but as a rule this will be more than offset by the better working of the system.

This system is especially recommended for high buildings and for mills and factories (see p. 1162).

Second. Ordinary One-pipe System, or "One-pipe Basement System."—In this system one large steam-main runs around the basement to a point where the last radiator or riser is taken off and is then connected into a return main, which conveys the water of condensation back to the boiler, or if there is no occasion for dropping the return below the basement floor, the steam main is continued around the basement and connected to the return in the back of the boiler.

The steam-main when it leaves the boiler is elevated close under the ceiling, and is graded down from the boiler about ½ in. in 10 ft., so that the water of condensation will flow towards the return. In this system as in the Mills system there is only one connection made to each direct radiator, which is an advantage over the double-pipe system, as there is only one valve to open or close in turning on or shutting off a radiator. Unlike the

* Prof. Carpenter.

Mills system, however, the steam and water flow in opposite directions in the risers. With this system a good automatic air-valve should be placed on the extreme end of the horizontal return main, above the water-line, to allow the escape of air that cannot escape through the radiators.

This method of piping is the one now used most extensively and when correctly installed gives good satisfaction.

Third. The Two-pipe System.—This system consists in having steam and return mains in the cellar and two connections to each radiator. The steam-main is graded down from the boiler about ½ in. in 10 ft., and is reduced in size as radiator or riser connections are taken off; at the end it is connected into the return main below the water-line. The return main increases in size as it goes towards the boiler, as connections are made to it from risers or radiators. Each radiator receives steam from a riser or connection taken from the steam-main and empties into the return through a return riser or connection, so that there is a complete circulation throughout the entire system.

This system was used almost exclusively twenty or thirty years ago, but is now confined mainly to large buildings and to buildings heated by indirect radiation.

Indirect Radiators must always have a flow and return pipe, and when used in buildings heated by the one-pipe system the return riser must be entered into a return main below the water-line.

The two-pipe system is naturally much more expensive than the one-pipe system, because twice as many radiator valves are required for the former and 50 to 75 per cent. more piping.

The Paul System of Heating.*—This is a patented system of exhausting all air from the radiators and piping, so that the steam circulates below or a little above atmospheric pressure. This is accomplished by attaching a patented air-valve to each radiator, and at any points where air might possibly connect on the returns, and connecting these valves by means of small air-pipes with an exhausting apparatus placed in the boiler-room. The valves are so constructed that while they permit of the passage of air no water can escape through them. The only difference between the Paul system and the ordinary single-pipe gravity system lies in exhausting

* See foot-note p. 1152.

the air so that the steam will be sucked through the pipes rather than forced.

The exhausting apparatus may be operated by steam, electricity, gas, or water, water being usually employed with low-pressure systems.

The cost of operating the exhausting apparatus when low-pressure boilers are used need not exceed 3 cents per day for a building containing 4,500 ft. of radiation. To install the system the steam-fitter must purchase the valves and exhausting apparatus from the Paul System Company and pay a small royalty, the amount depending upon the amount of radiation in the building. As by this system better circulation is provided than when the air discharges into the rooms through ordinary automatic air-valves the radiators are made more effective, consequently a little less radiation and smaller piping are required to do the same work. The cost of installation under the Paul system is, therefore, but little if any more than for the ordinary single-pipe gravity system, while it is claimed that the system will effect an economy of at least 20 per cent. in the amount of coal required for heating.

The system is in successful operation in a great many public and private buildings, and the company has agents in most of the larger cities from whom further information can be obtained. One great advantage of the system is that people in the rooms cannot tamper with the air-valves and there is no danger of their leaking.

Return of Water to Boiler in Non-gravity Systems of Steam Heating.—As stated on p. 1147, whenever the steam pressure in the radiators is less than that in the boiler, or when a radiator is placed below the water-line, then the water of condensation must be returned to a tank, called a receiver, placed below the lowest radiator, and returned from the receiver to the boiler by means of some mechanical device. As a rule, either a pump or a return trap is used for this purpose. For high-pressure systems, i.e., when steam is used to run machinery or to run the fan in a hot-blast system, a steam-pump running automatically is generally considered the most satisfactory device for returning the water to the boiler.

Where there is no engineer in constant attendance, a return trap will generally be preferable. The return trap works automatically and will return the water as well as a pump, besides being less expensive. The greatest objection to a return trap

seems to be that if it gets out of order from any cause, it is not as easily or quickly repaired as a pump.

A return trap should be placed upon or near the boiler and the bottom of the trap should be at least 2 ft. above the water-line of the boiler. A pump may be placed any distance below the water-line of the boiler and at a considerable distance from the boiler. In hot-blast heating the pump and receiver are generally placed near the heating-stacks and fan.

The Webster System * (controlled by Warren Webster & Co.).—This, like the Paul system, is a vacuum system of steam heating, but, unlike the Paul system, it exhausts all *water* of condensation as well as air, so that the flow pipes are at all times filled with dry steam. This system can also be applied to all classes of non-gravity heating apparatus and where exhaust steam is used. A Webster thermostatic water and air relief valve is placed on the drip end of each radiator and a small pipe connects each valve with the exhausting apparatus near the boiler. The water of condensation is taken to a receiver, from which it is fed back to the boiler. With this system comparatively small supply and return mains may be employed, but the radiation should, if anything, be increased.

This system is especially adapted to large heating plants, hot-blast systems, and dry kilns, and may be successfully and economically applied to a great variety of manufacturing processes by making slight modifications in its working details.

The patentees claim that the Webster system will give a better circulation and effect greater economy in maintenance than any other.

It has been successfully installed in a great many large buildings through the country and in many factories and manufacturing plants.

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* The vacuum system of heating was first introduced to the heating trade in this country some time in the past seventies by N. Y. Williams, a heating engineer of Philadelphia, Pa. His plan was to plug up all air-vents and to attach a pump to the main return pipe and exhaust all air and water from the steam-pipes, coils, and radiators in a system. The plan was an improvement to the many poorly constructed plants in use at the time, but it was not a complete success in itself. It would "short-circuit," i.e., the pump would act only on a portion of the system.

The Warren Webster Co. bought the inventor's rights and some other patents and in time introduced the Webster thermostatic valve, which is now used in all their work on all radiators, and has had much to do in making their system a success. The Paul and other vacuum systems have been introduced since.

Further information concerning this system may be obtained at any of the offices of the company.

Hot-blast System of Warming and Ventilating.—This system is used principally in buildings where a large amount of ventilation is required. The principle of the system is the forcing of large volumes of air over or through a heater and thence into the rooms to be warmed, and necessitates a fan for driving the air.

It may be successfully operated in connection with hot-air furnaces (see the author's work on "Churches and Chapels"), but, as a rule, the heat is furnished by steam-coils.

An ordinary hot-blast heating and ventilating plant consists of a steam-boiler, one or more stacks of steam-coils, a fan or fans driven either by a small steam-engine or electric motor, reducing-valve, receiver, and pump. The heating coils are usually collected in a "stack," over which all of the air for the building is passed, and from the stack the air is drawn or forced through hot-air pipes to all parts of the building. Direct radiation may also be employed in connection with this system for warming the halls and corridors or any rooms which do not require ventilation.

This system is especially adapted to the warming and ventilating of schools, churches, hospitals, and public buildings, and to many kinds of manufacturing plants. To insure successful results, however, it must be laid out with much care. Full information regarding it may be obtained from the American Blower Co., the Buffalo Forge Co., or the B. F. Sturtevant Co.

Pipe, Fittings, and Valves.—The pipe used for conveying steam or hot water was formerly made exclusively of wrought iron, but at the present time the term "wrought-iron pipe" is used merely to distinguish wrought from cast pipe. It is construed to mean merchant pipe, which is generally made from soft steel. Persons desiring *iron* pipe should specify "genuine wrought-iron pipe," for which an extra charge is made.

Up to the present time the pipe made of steel has not been as soft as that of wrought iron, and is often not so well welded and is more likely to split. Nevertheless, steel pipe is much more extensively used than the genuine wrought-iron pipe, although the latter is unquestionably the best.

Steam-pipe is put on the market in three grades, or thick-

nesses—*standard*, *extra strong*, and *double extra strong* (see tables of Wrought-iron Pipe, pp. 1205, 1206).

Each length of pipe as sold is provided with a collar or coupling (Fig. 18) on one end and has a thread cut on the other. Connections are made by screwing the threaded end of one pipe into the coupling on the other. Pipe is sold in random lengths varying from 16 to 24 ft. With the exception of couplings, the fittings used for connecting pipes and for giving them any desired direction with each other are made of cast and malleable iron.

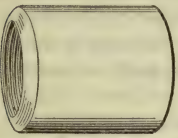


Fig. 18
Wrought-iron Coupling.

For use on heating pipes, cast-iron fittings are generally to be preferred to those of malleable iron, for several reasons (see Carpenter, p. 92).

Fittings for Joining Pipes.—For joining pipes in the same straight line, so as to make a continuous pipe from end to end, the coupling, Fig. 18, with right-hand threads cut in both ends is commonly used. With right-hand couplings it is impossible to disconnect the pipe at any place without commencing at the farther end and disconnecting the pipe section by section. Reducing couplings are made for uniting pipes of different sizes.

To connect two lengths of pipe, so that they can be disconnected at that point without interfering with other joints, three kinds of connections are in use:

(1) *Right and Left Couplings.*—The most common fitting for joining pipes 2 ins. diameter and under. It requires, however, that there shall be room for end motion of one of the pipes sufficient to insert it.

(2) *Lip Unions.*—These are generally used on pipes up to $1\frac{1}{2}$ or 2 ins. in diameter where it is desirable to have a joint that may be readily disconnected. The union consists of three pieces; two of these parts screw on to the ends of the pipe and are drawn together by a revolving collar which engages with the thread on one of the pieces, as shown by Fig. 19.

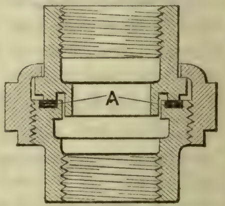


Fig. 19
Lip Union.

With this connection no appreciable play is required in the piping.

Unions are now commonly used in connecting radiators, the union being attached to the radiator valve.

(3) *Flange Unions* (Fig. 20).—These are used on pipes exceeding 2 ins. in diameter. The two parts of the union are first screwed to the pipes and then bolted together. A ring of packing must be placed between the flanges to make a tight joint.

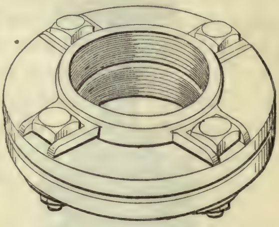


Fig. 20
Flange Union.

Nipples (Fig. 21) are frequently used in steam fitting for connecting pipes, radiators, and sectional boilers. They are made with right thread on both ends and right thread on one end and a left thread on the other.

Push nipples are made with ends bevelled and ground perfectly true, so as to make a tight joint by contact of the metal. Their use is confined to radiators and sectional boilers.

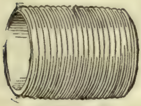
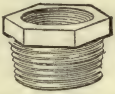


Fig. 21
Close Nipple.



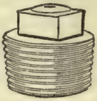


Fig. 22
Bushing. Plug.

Bushings are used for reducing the size of opening in a fitting. *Plugs* are used for closing the end of a fitting and *caps* for covering the end of a pipe.

A great variety of cast-iron fittings are carried in stock, such as elbows, tees, crosses, branch tees, Y bends, return bends, etc., each of these being made in a great variety of sizes and shapes.

A description of them may be found in the catalogues of dealers in steam-fitters' supplies.

Valves and Cocks.—Three classes of valves are used in steam-fitting, viz., globe valves, gate-valves, and check-valves.

The valve shown by Fig. 26 is a globe valve, but is commonly designated as an angle-valve, the term globe valve being commonly restricted to those valves which go on a straight line of pipe.

In the gate-valve the disc which closes the opening is at

right angles to the pipe. The gate-valve when open offers less obstruction to the flow of steam or water, and for this reason is largely used on water-pipes. Some steam-fitters con-

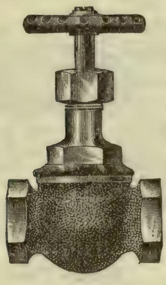


Fig. 23
Brass Globe Valve.

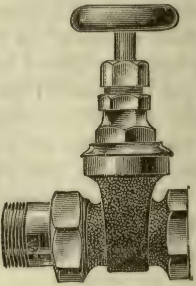


Fig. 24
Brass Gate-valve with Union.

tend that a gate-valve should not be used for steam, except on the main return, near the boiler.

A *disc valve* is commonly a globe or angle valve with a composition disc or ring similar to the washer on a compression cock, which fits against the seat of the valve. A Jenkins disc is a valve in which the disc or the entire valve is made by

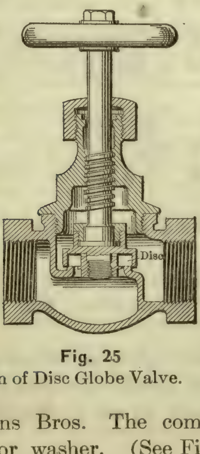


Fig. 25
Section of Disc Globe Valve.

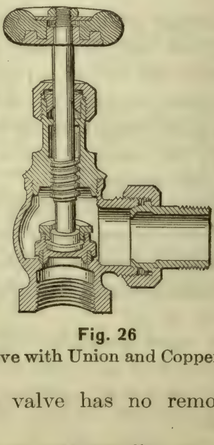


Fig. 26
Angle-valve with Union and Copper Disc.

Jenkins Bros. The common globe valve has no removable disc or washer. (See Fig. 27.)

Disc valves should always be used on steam-radiators.

A union valve is a globe or angle valve with a union on one side of the valve.

Globe valves are made for screw, union, or flange connections, although the latter is commonly used only on large pipes.

Globe and angle valves for 2-inch pipes and under are commonly made with brass bodies and either iron or wood handles. The larger sizes are commonly made with iron bodies. Radiator valves should have brass bodies and wood wheels.

When it is desired that radiators shall not be under the control of the occupants of the room, valves operated by a key may be used. Hot-water radiator valves may also be had with pedal attachments so that they may be opened or closed with the foot.

Special forms of quick-opening valves are largely used on hot-water radiators.

Obstruction to Flow Offered by Globe Valves.— When globe valves are placed on horizontal steam-mains, the stem should always be placed in a horizontal position, for if set vertically the seat of the valve forms an obstruction sufficient to fill the pipe at least half full of water (as shown by Fig. 27). Because of the obstructions which they offer to the flow of water, globe valves should not be used on hot-water pipes.

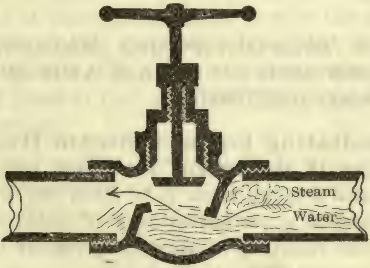


Fig. 27

Check-valves.—Where it is necessary that the flow shall always take place in one direction and there is danger of a reverse flow a check-valve must be employed. A check-valve is always required on the water-supply to a steam-boiler and on all connections to high-pressure boilers below water-line except the blow-off pipes.

Check-valves are of three kinds, the more common form

being that shown by Fig. 28, which has a valve which slides up and down. The swinging check-valve, Fig. 29, is also commonly employed. The third kind utilizes a ball in place of the sliding valve for closing the opening. The ball check-valve, however, is not much used in steam fitting.

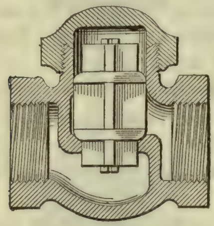


Fig. 28
Common Type of Check-valve.

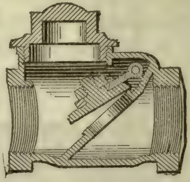


Fig. 29
Swing Check-valve.

A *cock* operates by means of a turned plug which has one or two holes bored transversely to its axis. When the plug is turned so that the hole is in line with the pipe the water flows through, and when the plug is turned the water is shut off.

Cocks are not much used in steam fitting, except on the blow-off pipe.

RULES FOR PROPORTIONING RADIATING SURFACE, AND SIZE OF STEAM AND HOT-WATER MAINS AND RETURNS.

Direct Radiating Surface—Steam Heating.—The common practice of determining the direct radiating surface required to warm a given room is to allow one square foot of radiating surface to a certain number of cubic feet of space contained in the room. The divisors given in the following table fairly represent current practice.

To find square feet of direct radiation required divide the cubic contents of room by the following factors:

For Dwellings	Divide by
Living-rooms, one side exposed	60 to 80
Living-rooms, two sides exposed	50 to 60
Living-rooms, three sides exposed	40 to 45
Sleeping-rooms	50 to 70
Halls and bathrooms	40 to 50

For Public Buildings	Divide by
Offices	50 to 75
Schoolrooms	50 to 70
Factories and stores	80 to 125
Assembly halls and churches	100 to 150

In buildings of more than two stories the first and top stories require the same amount of radiation if used for the same purpose, but the radiation in intermediate stories may be slightly reduced.

City houses require less heat than country houses and brick houses less than wooden houses.

Baldwin's Rule.—Mr. William J. Baldwin, in his excellent work on "Steam-heating for Buildings," * recommends the following rule, which he has used for several years, and which is not wholly empirical:

"Divide the difference in temperature between that at which the room is to be kept and the coldest outside atmosphere by the difference between the temperature of the steam-pipes and that at which you wish to keep the room and the quotient will be the square feet, or fraction thereof, of plate or pipe surface to each square foot of glass or its equivalent in wall surface." †

The equivalent glass surface is found by multiplying the superficial area of the walls in square feet by the number opposite the substance in the following table and dividing by 1,000 (the value of glass). The result is the equivalent of so many square feet of glass in cooling power and should be added to the window surface.

TABLE OF POWER OF TRANSMITTING HEAT OF VARIOUS BUILDING SUBSTANCES COMPARED WITH EACH OTHER.

Window glass	1,000
Oak and walnut	66
White pine	80
Pitch pine	100
Lath and plaster	75 to 100
Common brick (rough)	200 to 250
Common brick (whitewashed)	200
Granite or slate	250
Sheet iron	1,030 to 1,110

* Published by John Wiley & Sons, of New York.

† It should be noticed that this proportion *does not depend* upon the size

It must be distinctly understood that the extent of heating surface found in this way offsets only the windows and other cooling surfaces *it is figured against*, and does not provide for cold air admitted around loose windows or between the boarding of poorly constructed wooden houses. These latter conditions, when they exist, must be provided for by additional heating surface.

EXAMPLE 1. What amount of heating surface should be supplied to the sitting-room of a wooden dwelling with two outside walls, one 14 ft. by 9 ft. high and the other 15 ft. by 9 ft., the total window area being 54 sq. ft., the external temperature frequently being at 0° F., and the steam never exceeding 5 lbs. pressure?

Ans.—Temperature of room, $70° - 0° = 70°$; temperature of steam-pipes at 5 lbs., $228 - 70° = 158$; $70 \div 158 = .443$, or a little less than one half a square foot of heating surface to each square foot of glass or its equivalent.

Area of outside walls $= 14 \times 9 + 15 \times 9 = 126 + 135 = 261$. Subtracting the glass area, 54, we have 207 sq. ft. of lath and plaster.

$$\begin{array}{r} 207 \times \quad 100 = 20{,}700 \\ 54 \times 1{,}000 = 54{,}000 \\ \hline 1{,}000)74{,}700 \\ \hline \end{array}$$

Equivalent glass area = 74. Multiplying this by .443, we have 33 as the number of square feet of radiating surface required to warm the room, or 1 ft. of surface to 58 cu. ft. of air space.

*Rule of F. Schumann.**—"Divide the cubic feet of space of the room to be heated, the square feet of wall surface, and the square feet of the glass surface by the figures given under these headings in the following table and add the quotients together; the result will be the square feet of radiating surface required.

For the above example with northwest and southeast exposures and steam at 3 lbs. pressure this rule would require 26.7 sq. ft. of radiating surface for a change of once per hour and 45.1 sq. ft. for a change of twice per hour.

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of the room, but only upon the climate, pressure of the steam, and desired temperature of the room.

* Kent, p. 536.

SPACE, WALL, AND GLASS SURFACE WHICH ONE SQUARE FOOT OF RADIATING SURFACE WILL HEAT.

Air Change.	Steam Pressure in Pounds.	Space in Cubic Feet.	Exposure of Rooms.					
			All Sides.		Northwest.		Southeast.	
			Wall Surface. Sq. Ft.	Glass Surface. Sq. Ft.	Wall Surface. Sq. Ft.	Glass Surface. Sq. Ft.	Wall Surface. Sq. Ft.	Glass Surface. Sq. Ft.
Once per hour	1	190	13.8	7	15.87	8.05	16.56	8.4
	3	210	15.0	7.7	17.25	8.85	18.00	9.24
	5	225	16.5	8.5	18.97	9.77	19.80	10.20
Twice per hour	1	75	11.1	5.7	12.76	6.55	13.22	6.84
	3	82	12.1	6.2	13.91	7.13	14.52	7.44
	5	90	13.0	6.7	14.52	7.60	15.60	8.04

Prof. R. C. Carpenter says that for residences it is safe to assume that the air of the principal living-rooms will change twice in an hour, that of the halls three times, and that of the other rooms once per hour under ordinary conditions.

Prof. Carpenter, in his work on "Heating and Ventilating Buildings," gives the following formula, which is convenient and probably as accurate as any for general purposes,

$$h=\frac{N}{55}C+G+\tfrac{1}{4}W,$$

in which W = wall surface, G = glass or window surface, both in square feet, C = contents of room in cubic feet, N = number of times air will be changed per hour, and h = total heat-units required per degree of difference of temperature between the room and the surrounding space.

Under ordinary conditions of pressure and temperature one square foot of steam-heating surface will supply 280 heat-units per hour and 1 sq. ft. of hot-water heating surface 175 heat-units per hour.

To heat the room to 70° F. when the outside temperature is at zero, the square feet of direct radiating surface required will be $\frac{1}{4}h$ for steam heating and $\frac{4}{10}h$ for hot-water heating. For churches and auditoriums N should be taken at least equal to 3.

In Example I we have C = 1,890, W = 207, and G = 54. Hence h = 140 when the air is changed once per hour and 174

when changed twice per hour. The steam radiating surface required will be 35 and $43\frac{1}{2}$ sq. ft. respectively.

In practical work it is well to determine the heating surface by two or more rules and then use the larger quantity. Different localities and different grades of buildings also affect the amount of radiating surface required, so that practical steam-fitters are usually governed to some extent by their experience. There can never be any bad results from having an excess of heating surface provided all rooms have their proportionate amount, while a deficiency will always result in cold rooms in extremely cold weather.

Overhead Steam-pipes (A. R. Wolff, Stevens Indicator, 1887).—When the overhead system of steam heating is employed, in which system direct radiating pipes, usually $1\frac{1}{4}$ in. in diameter, are placed in rows overhead suspended upon horizontal racks, the pipes running horizontally and side by side around the whole interior of the building, from 2 to 3 ft. from the wall and from 2 to 4 ft. from the ceiling, the amount of $1\frac{1}{4}$-inch pipe required, according to Mr. C. J. H. Woodbury, for heating mills (for which this system is deservedly much in vogue) is about 1 ft. in length for every 90 cu. ft. of space. Of course a great range of difference exists due to the special character of the operating machinery in the mill, both in respect to the amount of air circulated by the machinery and also the aid to warming the room by the friction of the journals. For this system of radiation the Mills system of piping should be used.

Direct Radiation—Hot-water Heating.—*Rule of Thumb.*—Divide the cubic contents of room in cubic feet by the following factors; the result will be the square feet of radiation required:

For Dwellings	Divide by
Living-rooms, one side exposed	40 to 50
Living-rooms, two sides exposed	30 to 40
Living-rooms, three sides exposed	20 to 25
Sleeping-rooms	30 to 50
Halls and bathrooms	20 to 30

For Public Buildings	Divide by
Offices	30 to 40
Schoolrooms	30 to 40
Factories and stores	40 to 60
Assembly halls and churches	60 to 100

Prof. Carpenter's rule for direct hot-water radiation is the same as for steam (p. 1161), using 0.4 for a multiplier instead of $\frac{1}{4}$.

For Direct-indirect Radiation it is customary to allow 25 per cent. more surface for steam and $33\frac{1}{3}$ per cent. for hot-water than would be required for direct radiation.

Indirect Radiating Surface (Prof. Carpenter's Rule). —The radiating surface for indirect heating may be found by adding together the glass surface in the room to be warmed, one fourth the exposed wall surface, both in square feet, and multiplying by the following factors:

	Steam Heating.	Hot-water Heating.
First story	0.7	1.05
Second story	0.6	0.9
Third story	0.5	0.8

The total amount of incoming air which this amount of radiation will warm *per hour* in cubic feet may be found approximately by multiplying the radiating surface by the following factors:

	Steam Heating.	Hot-water Heating.
First story	200	125
Second story	250	160
Third story	300	200

If a greater quantity of air is required for ventilating purposes, an additional foot of heating surface should be allowed for each 250 cu. ft. of air heated by steam, or for each 150 cu. ft. heated by hot water.

For rooms which are specially exposed these results should be increased about 10 per cent., and 10 per cent. if the rooms are heated during the daytime only.

Size of Air-ducts and Registers for Indirect Radiator Stacks (Steam Heating).

For computing the area of duct from stack to the room and outlet pipe from the room the following data by Prof. Carpenter will probably give as good results as can be obtained by any rule, except where there is a very large glass area.

Rule.—Multiply the sum of the glass surface and one quarter the wall area by the appropriate factor given in the following table:

TABLE OF FACTORS FOR AREA OF AIR-FLUES.

Story of Building.	Supply Duct.			Ventilating Duct.		
	Approximate Head in Feet.	Velocity in Feet per Second.	Factor for Area. Sq. Ins.	Approximate Distance to Roof.	Velocity in Feet per Second.	Factor for Area. Sq. Ins.
First floor.	5	2.8	2.40	47	5.5	0.93
Second floor.	28	6.8	0.95	32	4.2	1.27
Third floor.	40	8.1	0.82	20	3.6	1.33
Fourth floor.	50	9.0	0.71	10	2.6	2.17

The cold-air, or out-door, supply to the stack should have a sectional area equal to about three fourths of that of the warm-air flue.

The nominal size of registers should be about 50 per cent. greater than the area of the warm-air flue.

The following sizes for air-ducts and registers for indirect steam radiation are published by the International Heater Co., and a similar table is given on p. 1133. It is evident that some judgment must be used with all three tables.

Square Feet of Radiation.	Cold-air Duct to Stack.		Warm-air Duct.		Registers.		Tapping.
	For First Floor.	For Upper Floors.	For First Floor.	For Upper Floors.	First Floor.	Upper Floors.	
	Sq. Ins.	Sq. Ins.	Sq. Ins.	Sq. Ins.	Ins.	Ins.	Ins.
50	50	40	75	50	10×12	8×10	$1 \times \frac{3}{4}$
60	60	45	90	60	10×14	8×12	$1\frac{1}{4} \times 1$
70	70	50	105	70	12×15	10×12	$1\frac{1}{4} \times 1$
80	80	60	120	80	12×15	10×12	$1\frac{1}{4} \times 1$
90	90	70	135	90	12×19	10×14	$1\frac{1}{2} \times 1\frac{1}{4}$
100	100	75	150	100	12×19	12×15	$1\frac{1}{2} \times 1\frac{1}{4}$

Size of Steam-mains and Return Pipes.

Mr. George H. Babcock gives the following rule for gravity heating systems with separate returns (two-pipe system):

"The diameter of the steam-mains leading from the boiler should be equal in inches to one tenth the square root of radiating surface, mains included, in square feet."

If the mains are covered they may be neglected in figuring radiating surface.

For the one-pipe basement system it will be safe to use one ninth instead of one tenth in the above rule, unless the pipes are very long. It is always better to have the mains larger than is necessary rather than too small.

Steam-mains should never be less than 1½ ins. in diameter.

The sizes of returns that will prove satisfactory for given sizes of steam-mains are given by Prof. Carpenter as follows, no return to be less than 1 in. diameter:

Diameter Steam-pipe.	Diameter Return Pipe.	Diameter Steam-pipe.	Diameter Return Pipe.
Inches.	Inches.	Inches.	Inches.
1½	1	5	2½
2	1¼	6	3
2½	1¼	8	3½
3	1½	9	4
3½	1½	10	4½
4	2	12	5

For connecting direct radiators with the *single*-pipe system, the following sizes of pipes should be used:

For radiators containing 24 sq. ft. or under, 1-inch pipe; for radiators containing 24 to 60 sq. ft., 1¼-inch pipe; for radiators containing 60 to 180 sq. ft., 1½-inch pipe; for radiators containing above 100 sq. ft., 2-inch pipe.

For Two-pipe Work.—Radiators containing 48 sq. ft. and under, 1-inch supply, ¾-inch return; 50 sq. ft. to 96 sq. ft, 1¼-inch supply, 1-inch return; above 96 sq. ft., 1½-inch supply, 1¼-inch return.

For Indirect Heating it will usually be sufficiently accurate to use a pipe whose diameter is 1.4 times greater than that for direct heating.*

* Prof. Carpenter.

For Hot-water Heating.—Prof. Carpenter says: "We may take as a practical rule, applicable when the pipes are less than 200 ft. in length: The diameter of main supply or main return pipe in a system of direct hot-water heating should be one-pipe size greater than the square root of the number of square feet of radiating surface divided by 9 for the first story, by 10 for the second story, and by 11 for the third story of a building; for indirect hot-water heating multiply above results by 1.5."

In hot-water heating the return pipe must have the same diameter as the supply pipe, and the capacity of both should be equal to total capacity of risers. For equalizing hot-water pipes the tables on p. 1203 will be found very convenient.

The standard tapping for hot-water radiators is as follows: Radiators containing 40 sq. ft. and under, 1 in.; above 40 but not exceeding 72 sq. ft., $1\frac{1}{4}$ ins.; above 72 sq. ft., $1\frac{1}{2}$ ins.

Boiler.—To find the size of boiler necessary to supply any given amount of radiation, see p. 1146.

Covering of Pipes.

Steam and hot-water mains radiate more heat in proportion to their surface than do the radiators which they supply, and unless this heat is needed for warming the space through which the pipes pass, it represents a very material loss in the consumption of fuel.

To reduce this loss to a minimum, it is customary to cover all pipes in unfinished basements with some insulating substance. The saving in fuel effected by a good covering will more than pay for its cost in a few seasons.

"The best insulating substance known is air confined in minute particles or cells, so that heat cannot be removed by convection. No covering can equal or surpass that of perfectly still and stagnant air; and the value of most insulating substances depends upon the power of holding minute quantities in such a manner that circulation cannot take place. The best known insulating substance is a covering of hair-felt, wool, or eiderdown, each of which, however, is open to the objection that, if kept a long time in a confined atmosphere and at a temperature of 150° or above, it becomes brittle and partly loses its insulating power.

"A covering made by wrapping three or more layers of asbestos paper, each about $\frac{1}{16}$ in. thick, on the pipe, covering with a layer of hair-felt $\frac{3}{4}$ in. in thickness, and wrapping the

whole with canvas or paper is much used. This covering has an effective life of about five years on high-pressure steam-pipes and ten to fifteen years on low-temperature pipes. There are a large number of coverings regularly manufactured for use in such a form that they can be easily applied or removed if desired. There is a very great difference in the value of these coverings; some of them are very heavy and contain a large amount of mineral matter with little confined air and are very poor insulators. Some are composed entirely of incombustible matter and are nearly as good insulators as hair-felt. In general, the value of a covering is inversely proportional to its weight, the lighter the covering the better its insulating properties; other things being equal, the incombustible mineral substances are to be preferred to combustible material. The table on the next page gives the results of some actual tests of different coverings, which were conducted with great care and on a sufficiently large scale to eliminate slight errors of observation. In general, the thickness of the coverings tested was 1 in. Some tests were made with the coverings of different thicknesses, from which it would appear that the gain in insulating power obtained by increasing the thickness is very slight compared with the increase in cost. If the material is a good conductor its heat-insulating power is lessened rather than diminished by increasing the thickness beyond a certain point." *

Sectional Coverings.—It may be seen from this table that magnesia, asbestos, and mineral wool are the three materials most valuable for the covering of steam-pipes, as wool and hair, although being better non-conductors, are short-lived on steam-pipes. Wool covering is extensively used, however, on hot-water pipes. Sectional coverings, moulded and formed to fit different sizes of pipes, are made by many parties, and are used almost exclusively for covering steam, and to a large extent for hot-water, pipes. After the sections are applied they are commonly secured by brass lacquered bands. The fittings, such as elbows and tees, are usually plastered with plastic asbestos or magnesia and then covered with canvas applied with flour paste.

The foregoing data, in connection with the following table, will enable the reader to judge which kind of covering is likely to be the most effective.

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* Prof. Carpenter in "Heating and Ventilating Buildings."

TESTS OF VARIOUS PIPE-COVERINGS MADE AT SIBLEY COLLEGE, CORNELL UNIVERSITY.

Kind of Covering.	Relative Amount of Heat Transmitted.
Naked pipe	100.0
Two layers asbestos paper, 1 in. hair-felt, and canvas cover	15.2
Two layers asbestos paper, 1 in. hair-felt, canvas cover, wrapped with manilla paper	15.0
Two layers asbestos paper, 1 in. hair-felt	17.0
Hair-felt sectional covering, asbestos lined	18.6
One thickness asbestos board	59.4
Four thicknesses asbestos paper	50.3
Two layers asbestos paper	77.7
Wool felt, asbestos lined	23.1
Wool felt with air spaces, asbestos lined	19.7
Wool felt, plaster-of-Paris lined	25.9
Asbestos moulded, mixed with plaster of Paris	31.8
Asbestos felted, pure long fibre	20.1
Asbestos and sponge	18.8
Asbestos and wool felt	20.8
Magnesia, moulded, applied in plastic condition	22.4
Magnesia, sectional	18.8
Mineral wool, sectional	19.3
Rock wool, fibrous	20.3
Rock wool, felted	20.9
Fossil meal, moulded, ¾ inch thick	29 7
Pipe painted with black asphaltum	105.5
Pipe painted with light drab lead paint	108.7
Glossy white paint	95.0

Hot-water Heating.—The system of heating by hot water consists of circulating hot water in the radiators instead of steam. The boiler, pipes, and radiators are completely filled with water, the flow or circulation pipes being attached to the top of the boiler and the return pipes to the bottom; the water in the boiler when heated rises and circulates through the pipes and radiators, parts with a portion of its heat, thus becoming colder and heavier, and passes down through the return pipes to the boiler, where it is again heated.

There are two general systems of hot-water heating, viz.,

(1) the open-tank system, and (2) the closed-tank, or pressure, system.

With the open-tank system an open expansion tank is connected to the heating system in such a way as to receive the increase in the volume of the water due to expansion by heat, and is connected with the outside air by a vent pipe, so that there is no pressure on the tank. Fig. 30 shows the common type of expansion tank, although copper-lined wooden tanks with automatic supply pipe, similar to W. C. tanks, are sometimes used.

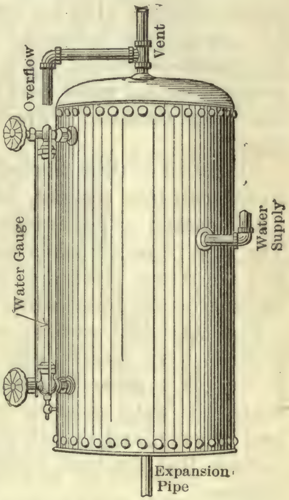

Fig. 30
Expansion Tank.

With the pressure system a similar tank is used, but the vent pipe is closed and a safety-valve, which will open when the pressure reaches a certain point, is placed on the overflow pipe. By increasing the pressure on the system, the water may be heated up to the temperature of low-pressure steam, and hence less radiatin surface and smaller pipes may be used.

The open system is most generally used, although the closed system is used occasionally.

The closed system is always open to the danger of a serious explosion from the safety-valve becoming inoperative or from the giving away of any part of the apparatus. This system cannot be recommended for house heating.

With the open expansion tank, about the only chance for an explosion is by the stopping of the expansion pipe, either through freezing or by the closing of a valve in the pipe. To avoid this, *no stop or valve* should be placed on the expansion pipe, and the expansion pipe should be well protected from frost.

The expansion pipe is usually taken off from the supply to one of the radiators in the upper story and the tank should always be on a level at least 2 or 3 ft. above the highest radiator.

The capacity of the tank should be somewhat greater than

one twentieth of the total cubical contents of heater, pipes, and radiators.

Boiler and Radiators.—Hot-water radiators have the same appearance as steam-radiators, but as a rule there is a slight difference in the interior to improve the circulation.

Almost any boiler that is suitable for steam heating can be used for hot-water heating, and most of the sectional boilers mentioned on p. 1144 are used for both kinds of heating. For hot water, the safety-valve and water-gauge are omitted. For residence heating, a great variety of small boilers especially designed for hot water have been placed on the market, notably the "Ideal Portable," "Spence," "Gurney, 400 series," "Palace King."

Nearly all of these heaters are made up of a number of horizontal cast-iron sections, which are bolted together and the joints packed or push-nipples used to make them water-tight. The flow pipes are taken from the top of the upper section, and the return pipes are connected with the lowest section, which generally forms either the fire-pot or the ash-pit.

The successful working of a hot-water heating apparatus depends very largely upon the proper construction of the boiler. It is generally admitted that in an efficient hot-water heater the water must be cut up into small portions, so as to heat quickly, and the whole arrangement of the heater should be such that the least possible resistance is offered to free circulation.

The boiler in which the most powerful circulation is maintained with the least consumption of fuel is the most satisfactory as well as the cheapest.

The method employed in connecting the joints and the facilities for cleaning fire surfaces are also points that should be carefully examined.

For the capacity of the various sizes and styles of heaters the architect or owner must depend largely upon the tables given by the manufacturers.

A hot-water apparatus is generally filled by connecting the house supply to return pipe at or near the heater. Sometimes a supply is connected with the expansion tank and a ball cock placed on it to insure that there shall always be three or four inches of water in the tank. At the lowest point of apparatus a draw-off, or emptying-cock, should be placed, to empty the system at any time.

The apparatus should be kept *full of water* during the summer months. This excludes the air and prevents corrosion or oxidation of pipes.

System of Piping.—Three systems of hot-water piping are in vogue, corresponding to the three systems described for steam heating:

(1) The overhead system, in which the hot water is first conducted to the highest part of the building, usually to the attic, and from thence distributed to the radiators by return pipes, exactly as in the Mills system, except that with hot water a top and bottom connection is made with each radiator, the water flowing into the radiator at the top and out at the bottom.

An improvement on this system is to have a separate return for the radiators as in the two-pipe system.

(2) *Two-pipe System.*—This is the system most commonly used. "In this system the mains and distributing pipe have an inclination *upward* from the heater; the returns are parallel to the main and have an inclination downward toward the heater, connecting at its lower part. The flow pipes are taken from the top of the main and supply one or more radiators. The return risers from the radiators are connected with the return pipe in a similar manner. In this system great care must be taken to produce nearly equal resistance to flow in all branches leading to the different radiators. It will be found that invariably the principal current of heated water will take the path of least resistance, and that a small obstruction, any inequality in piping, etc., is sufficient to make very great differences in the amount of heat received in different parts of the same system. For instance, two branch pipes connected at opposite ends to a tee, which itself is connected by a centre opening to a riser, are almost certain to have an irregular and uncertain circulation." *

Where indirect radiation is used in hot-water heating, the return pipe should be dropped below the floor and all return risers should be separately connected with the main return.

(3) *One-pipe System.*—In this system a single pipe is run around the basement as in the one-pipe steam system, except that the main hot-water pipe rises from the boiler; the flow pipes are taken from the top of the main and the water after

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* Prof. Carpenter.

passing through the radiators is returned by a separate pipe which is connected with the bottom of the main. With this system the water in the main is chilled wherever the returns are connected with it, so that the radiators at the far end of the system cannot be heated to as high a temperature as those which receive the water as it comes from the boiler.

A larger main is required for this system than for system No. 2. For small jobs, and particularly with boilers with horizontal sections, this system may be made to work satisfactorily, but the two-pipe system is always to be preferred.

For hot-water heating, special fittings are made which insure a more positive circulation than the ordinary fittings used in steam piping.

Rules for computing radiating surface, diameter of pipes, etc., are given on pp. 1162, 1163, and 1166.

Comparative Advantages and Disadvantages of Steam and Hot-water Heating.

(1) *Safety.*—An open-tank hot-water system with no valve on the expansion tank cannot possibly explode unless the expansion pipe should freeze, which is quite unlikely.

With steam gross carelessness may cause an explosion, although explosions of gravity heating plants are quite rare.

(2) *Comfort.*—There is probably little difference in this respect between steam and hot water, if both are well designed. Hot-water radiators do not become as hot as steam-radiators, and it is claimed that for this reason they do not dry, or "scotch," the air as much as steam-radiators, and therefore hot-water heating must be healthier.

The heat of a hot-water apparatus can be perfectly controlled by either the fire in the heater or the valve on the radiator, by partly closing it; whereas with steam-radiators the valve must be wide open or tightly closed. Also, with a hot-water apparatus, some of the radiators may be run at their full capacity, while others may be partly or entirely shut off without causing noise or in any way interfering with the perfect working of the system.

A hot-water apparatus is perfectly noiseless in operation, there being none of the snapping or gurgling noises common with steam.

(3) *First Cost.*—On an average, a hot-water apparatus costs about one third more than a steam apparatus to do the same work. This is because the hot-water apparatus requires nearly twice as much radiating surface, larger piping, and more expensive fittings.

(4) *Economy in Running.*—With a steam-heating apparatus, no heat is given off unless the water is kept boiling, while hot-water radiators will give off heat with water in the boiler at a temperature of 100°, consequently in moderately warm weather a hot-water plant will generally keep the rooms comfortable with a less consumption of coal than a steam-heating plant. In very cold weather, when the heating apparatus is worked to its full capacity, there is but little difference, if any, in the amount of coal consumed for either steam or hot-water heating.

In considering statements as to the economy of different heating systems, it should be remembered that the economy of any heating apparatus depends largely on the way in which it is run or upon the party having charge of the plant.

Disadvantages of Hot-water Heating.—About the only objections that can be urged against hot-water heating are increased first cost, danger from freezing, extra space occupied by radiators, and the fact that a building cannot be as quickly warmed by hot water as by steam.

It is also more difficult to secure uniform circulation in a large hot-water plant than in a large steam-plant.

While in large buildings and those that are not kept warm all the time many of these objections are of considerable importance, they do not, as a rule, hold good in residences, which are kept at a uniform temperature and in which the extra size of the radiators is of little consequence.

The danger of freezing is very much greater with hot-water circulation than with steam, and on this account hot-water indirect radiation must be used with much caution.

Summary.—For a residence of eight, ten, or twelve rooms probably 90 per cent. of those who are familiar with both steam and hot-water heating would recommend hot water.

For larger residences and small apartment houses, about as many would recommend steam as hot water, and for still larger buildings, probably 90 per cent. of heating engineers would recommend a gravity steam system or either the "Webster" or "Paul" system.

Hot-air, Steam, and Hot-water Heating in Residences.

Much advancement has been made of late years in the methods of heating residences and in the apparatus intended for that purpose. While it is impossible in this book to treat the subject in detail, it is believed that the following information will be of value in deciding upon the kind of heating to be used, and in selecting an efficient apparatus and seeing that it is properly put in.

In deciding upon a heating apparatus for a dwelling, the governing conditions are, generally, (A) the size of the building, and (B) the limit of first cost. When the latter condition is not a controlling one, the cost of running the apparatus should be given the first consideration.

For residences of eight or ten rooms and covering not more than 1,200 sq. ft. of ground the author would recommend hot-air heating by means of a good furnace.

For residences covering 1,400 sq. ft., a combination hot-air and water system is recommended, or an entire hot-water system.

For still larger residences, a steam or hot-water apparatus should be used.

Furnace Heating.—For warming residences not exceeding 1,200 sq. ft. of ground area, the author believes a good furnace, properly set and with hot-air pipes of proper size, suitably located, will give the best satisfaction, as it is economical in first cost, easy to manage, costs little for repairs, and furnishes a pleasant and healthy heat at no greater expense of running than with steam or hot water.

The most common defects observed in furnace-heating are overheating of the air, vitiating of the air by the gases of combustion, and imperfect distribution of the heat.

The first two defects may be entirely avoided if sufficient care is exercised in the selection and setting up of the furnace and in tending the fire, and the last defect may be reduced to a minimum by a wise location and proper proportion of the flues and registers.

The cause of the unsatisfactory heating of a great many houses by furnaces is in the owner or builder refusing to pay the necessary price for a first-class furnace and for the best

workmanship and materials. The same carelessness and "skinning" that is sometimes permitted with furnace work, if permitted on a steam or hot-water apparatus, would in most cases prevent their working at all.

Furnace heating may be divided into two parts, the *production* of heat and the *distribution* of the heat.

The former depends entirely upon the furnace, its setting, cold-air supply, draught, kind of fuel, and attendance.

The Furnace.—In principle, a hot-air furnace is simply a stove or heater incased with iron or brick, so as to form an air chamber between the heater and casing. The air enters at the bottom of the chamber, passes over the heated surfaces of the heater, and is conducted by the hot-air pipes to the various rooms.

The external surface of the fire-pot and all portions of the heater which receive heat from the fire or smoke are called the *radiating* surface.

As a rule, the furnace which has the greatest radiating surface in proportion to the size of the fire-pot will give off the most heat for a given amount of fuel consumed.

As the amount of radiating surface largely affects the weight of a furnace, and the latter in a great measure the selling price, it is obvious that the best furnaces must cost the most. It is true that one furnace may have its radiating surfaces better arranged than another, so as to give off more heat for a less quantity of metal, but it is seldom that a very light furnace, particularly if of cast iron, is a good heater.

Furnaces should be so designed that the smoke, after leaving the combustion-chamber, must travel around the radiator one or more times before finding an exit to the chimney. With a chimney-flue of proper size and topped out well above the roof, it is possible to make the smoke travel a long distance and thus obtain great economy of fuel. The best furnaces are designed on this principle.

Besides having a large radiating surface, the furnace should have as few joints as possible, and should be arranged so as to be easily cleaned.

Furnaces are made of cast iron, wrought iron, and steel, either used singly or combined. The radiating surface above the fire-pot can be made more cheaply of wrought iron than of cast iron, and in certain arrangements it is just as serviceable.

While there are excellent furnaces made of wrought iron and

steel, the author believes that a heavy cast-iron furnace is the most durable, and can be made as tight. Some furnaces are made chiefly of cast iron, but with air or smoke flues of wrought iron fitting into cast-iron sockets. This arrangement is not generally approved, as the two metals expand and contract unequally, thus tending to open the joint.

There are so many styles of furnaces manufactured that it is quite impossible to go further into details. It may be said, however, that the furnace shown in Fig. 31, made by the Richard-

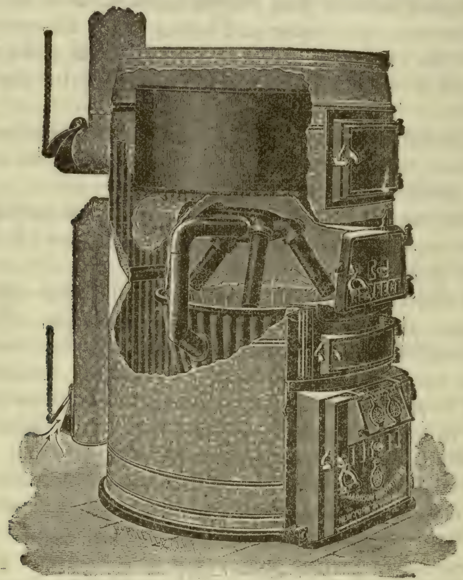

Fig. 31

son & Boynton Company, is representative of the best type of cast-iron furnace, and that shown in Fig. 32, made by Isaac A. Sheppard & Co., of a modern steel-plate furnace. Fig. 33, of which the Excelsior Steel Furnace Company are the makers, shows a type of furnace which consists of a plain combustion-chamber with a steel radiator. This radiator is divided with a horizontal partition, so that smoke must ciculate entirely around it before it enters the flue. This furnace is intended for soft coal. The more modern furnaces, constructed for burning

soft coal, have provision for the introduction of superheated air into the fire-box, thereby preventing the formation of soot and causing thorough combustion and intense heat. The one shown in Fig. 31 is a hot-air blast-furnace, and is supplied with oxygen at a high temperature for either hard or soft coal, accelerating and intensifying combustion to a very high degree.

In the *Twentieth Century furnace* the fire-pot contains cells and slots cast within the walls of the pot which admit air at twenty points equally distributed around the circumference of the same. By reason of this admission of air the fire burns from the top down and from the circumference toward the centre, causing an intense heat around the outside of the bowl. This furnace can be operated successfully with steam coal.

The Thatcher Furnace Company are makers of a tubular furnace that seems to possess considerable merit.

The casing surrounding the heater may be of brick or sheet iron. If of brick, it should consist of two 4-inch walls with a space between, the inner wall being generally built on a circle and the outer one on a square.

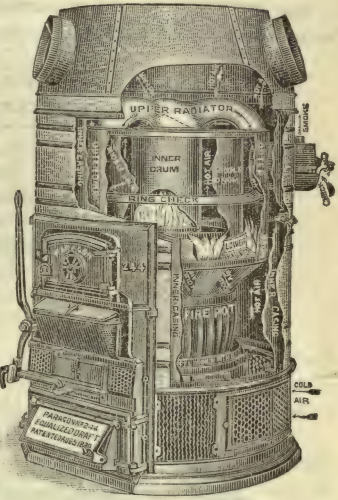

Fig. 32

"Brick set" furnaces are not as common as they formerly were, as they can be cased as well with iron and without occupying so much space in the cellar. When cased with sheet iron, the furnace is designated as "portable." Portable furnaces should always have a double casing with an inch space between. The inner casing may be of black iron, but the outer one should be galvanized. The hot air is thrown into the pipes better if the top of the casing is truncated, as in Fig. 32.

Cold-air Supply.—In a house heated by a furnace, the temperature of the rooms is maintained by a constant incoming current of hot air, and it is *absolutely* necessary for satisfactory heating that proper provision be made for supplying this air

to the furnace, and on no account should a hot-air furnace be used without being provided with a direct supply of air from outside the building. In dwellings this may be best accomplished by putting an opening in the external wall just beneath the first-floor joist and as far above the ground as the elevation of the building will permit. From this opening, which should be covered with galvanized wire netting of about three eighths of an inch mesh, a duct or flue should be carried to the air-pit *under* the furnace, as shown in Fig. 33.

The duct may be either carried horizontally under the basement ceiling until near the furnace and then dropped to the air-

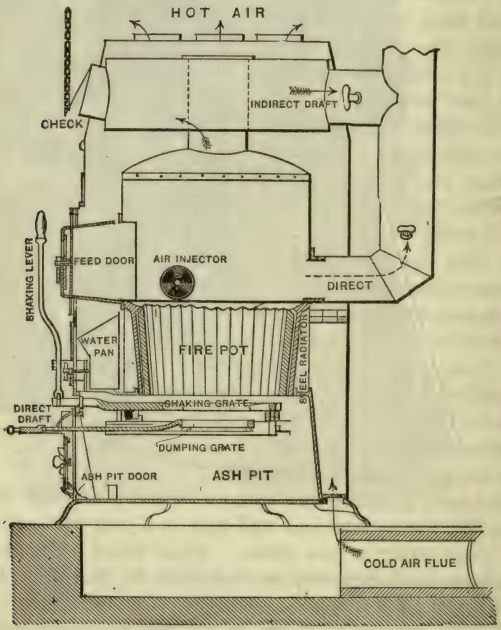

Fig. 33

pit, or it may be carried down against the cellar wall and thence under the floor to the furnace. The portion of the duct above the floor should be built of well-seasoned matched boards or of galvanized iron. The portion below the floor should be constructed either of stone, brick, or glazed tile, and should be

tightly cemented. If of brick or stone, the duct should be covered with stone slabs with the edges roughly dressed and the joints cemented. The air-duct should not be carried under the floor if the soil is at all damp, nor near any drain.

Fig. 34 shows the formation and construction of foundation and pit of a portable furnace.

Besides the external air supply, it is also a good idea to have a smaller air-duct leading from a register in the front hall to the base of the furnace. This duct may be of wood, tin, or

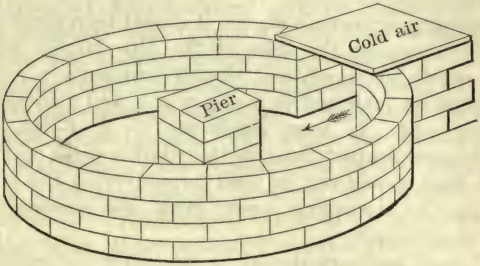

Fig. 34
Foundation and Pit of a Portable Furnace.

galvanized iron, and may be connected either with the base of the furnace above the floor or feed into the outside duct, but care should be taken to prevent the air from blowing from the outside duct up through the inside one.

An inside duct will produce a better circulation of air through the house, and on very cold nights the outside duct may be shut off and the air taken entirely from the front hall, as the air from this source, having nothing to contaminate it, will be reasonably pure.

The Hot-air Pipes and Registers.—The pipes which convey the heated air from the furnace to the various rooms should be of bright IX tin for sizes less than 14 ins. in diameter and of No. 26 galvanized iron for larger sizes.

All pipes below the basement ceiling should be round, and for the best work should be covered with asbestos paper, pasted to the pipe with a specially prepared paste.

The vertical hot-air pipes, to rooms in second or third stories, are frequently termed "stacks." They usually pass up between the studding of the partitions in the lower stories, thus necessitating a shallow pipe.

For medium- and low-cost houses the stacks are usually made $3\frac{3}{4}$ ins. deep, of one thickness of tin, and wrapped with asbestos paper pasted to the tin. For a better class of buildings double pipes are, or should be, used for the stacks. These stacks have an air space between the outside and inside pipes, affording a circulation of air, which makes the stacks absolutely safe, thus obviating the necessity of iron lath in front of the stack.

The table on p. 1197 gives the sizes and dimensions of safety double hot-air stacks made by the Excelsior Steel Furnace Company.

In providing for hot-air stacks, it should be remembered that the friction against the sides of the pipe largely affects the volume of air conveyed, and that consequently a round pipe is always to be preferred to a square one, and a square pipe to a shallow pipe. In large residences, 5- or 6-inch studding should be used for partitions, so that thicker pipes may be used.

Brick flues should not be used for conveying hot air, as the loss of heat by absorption is very great, and economical results cannot be obtained.

The hot-air registers should be set in double register boxes made of tin, and the bottom of the stacks should terminate in a "boot" or "footing," arranged in such a way as will insure the quick and easy flow of hot air from the feed-pipe into the stacks.

Warm-air Radiators.—In the use of warm-air furnaces it is oftentimes extremely difficult to heat rooms located at a distance from the furnace, rooms that are without any means of ventilation, or rooms which are greatly exposed to outside winds. This difficulty may sometimes be overcome by using a warm-air radiator placed over the outlet of the furnace pipe, which must be in the floor. These radiators are made of sheet steel and are so constructed that they set up a circulation of air in the room which tends to draw the air from the furnace. They somewhat resemble a direct-indirect steam-radiator.*

Ventilation.—A hot-air furnace plant, properly put in, will furnish a good supply of fresh air, and therefore afford fairly good ventilation, if means are provided for carrying off the foul air in the rooms. The warm air entering a room must of necessity force out an equal quantity of the air already in the room;

* A very good pattern of warm-air radiator is made by the International Heater Co.

exits are often found in the spaces around the doors and windows, but these are rarely sufficient to carry away the air as fast as it would enter if unimpeded. Fireplaces, especially if kept in use, afford excellent ventilation. A good arrangement for obtaining ventilation is by building a large flue in a central chimney and using a galvanized-iron smoke-stack, placed in the centre of it, for the furnace. The space surrounding the smoke-pipe may then be used for ventilation and ducts from different rooms connected with it.

Location of Furnace.—Upon the location of the furnace the successful heating of the house often depends, and it is a matter that requires careful consideration.

As a general rule, the furnace should be located in the basement, near the centre of the space occupied by the registers, and a little nearer the side from which the prevailing winds come in winter-time. The tendency, in hot-air heating, when the wind is blowing strong in severe cold weather, is for the rooms on the further side of the house from the wind to be overheated, while those against the wind are poorly heated, the registers on the windward side delivering almost no hot air. Therefore, to counteract this tendency, the furnace should be placed some few feet toward the windward side of the building, *provided* this does not make the pipes to the general, or family, living-rooms longer than the others.

The height of the basement should be such that the "leaders," or horizontal hot-air pipes below basement ceiling, may have a pitch of 1½ ins. per running foot upward from the furnace. If there is no inclination to these pipes, the first-story rooms will be heated with difficulty. For a residence of ten rooms the furnace-room should have a clear height of at least 7 ft. 6 ins.

Cold-air Opening.—If only one external cold-air supply is used, it should be taken from the direction from which the prevailing winds come. For buildings in exposed situations it is desirable to have a cold-air supply from the opposite side of the building also, the ducts connecting, and each being furnished with a damper, so that either duct may be used, according to the direction of the wind. Cases have been known where the wind blowing from the opposite direction of the cold-air supply has sucked the air from the house through the furnace and cold-air duct, thus actually reversing the natural operation of the furnace. Two supplies will obviate this possibility.

Location of Stacks and Registers.—To insure the best results, the location of furnace, stacks, and registers should be planned out before the work of construction begins, for while the building need not be planned to suit the heating apparatus, it almost always happens that the setting of the partitions, swinging of doors, and placing of studs and joists can be arranged so as to favor the placing of stacks and registers, without seriously affecting any desired arrangement of the plan, and this can be done much better on the plans than after the house is started.

It is generally conceded that the hot-air stacks should be placed in the partitions and as near to the furnace as practicable, and that all horizontal branches should be as short as possible, as the air travels much slower in the horizontal branches and more heat is lost from radiation. The registers should be placed as near the stack as possible; they should *not* be placed near the windows, nor where the doors will swing over or against them, nor in the floor near an open fireplace.

Whether the register shall be placed in the floor or partition is a matter that should be decided by the owner. It is claimed that the circulation from a wall register is not as good as from one placed in the floor, and the wall above the register generally becomes discolored after a time by the dust that is occasionally blown up through the pipes. On the other hand, floor registers catch much more dirt from sweeping the rooms, and many ladies object to having their carpets cut. The author believes that it is healthier to have the registers placed in the wall. Convex registers are to be preferred for walls, as they deliver more air than do the ordinary flat registers. It sometimes happens that the stacks must be put in an outside wall. When such is the case, the stack should be double and wrapped with asbestos paper as well. Stacks should *not* be placed in outside walls, however, when it is possible to avoid it.

Calculations for Size of Furnace, Pipes, and Registers.

There appears to be no rule by which the architect can determine the size of the furnace that should be used to heat a given building other than by using the tables given by the various manufacturers. Rules have been given for determining the necessary grate area of a furnace, but it is utterly impossible to make such a rule that will apply to all furnaces, as the heating

capacity depends almost as much upon the amount and character of the radiating surface, and these vary with the make of the furnace. Some manufacturers give rules which take into account not only the cubic space to be heated, but also the outside wall and the glass area, both of which should be considered in deciding on the size of the heater. Most furnace-makers, however, merely give the amount of cubic space that the different sizes of their particular furnaces will heat, and as there is no way of telling how reliable these figures are, except by experience, it is wise to have the contractor give a guarantee that the furnace shall heat the building to 70° in zero weather without forcing the furnace.

Pipes and Registers.—The tables given in various books and catalogues for the size of pipes and registers vary a great deal and must be used with considerable judgment. The following table appears to the author to be as reliable as any:

TABLE OF CAPACITY OF HOT-AIR PIPES AND REGISTERS.

Size of Register.	Equivalent in Round or Leader Pipe.	Equivalent in Square or Riser Pipe.	Cubic Feet of Space on First Floor Same Will Heat.	Cubic Feet on Second Floor.	Cubic Feet on Third Floor.
6× 8	6 in.	4× 8	400	450	500
*8× 8	7 "	4×10	450	500	560
*8×10	8 "	4×10	500	850	880
*8×12	8 "	4×11	800	1000	1050
*9×12	9 "	4×12	1050	1250	1320
*9×14	9 "	4×14	1050	1350	1450
*10×12	10 "	4×14	1500	1650	1800
*10×14	10 "	6×10	1800	2000	2200
10×16	10 "	6×10	1800	2000	2200
12×14	12 "	6×12	2200	2300	2500
*12×15	12 "	6×12	2250	2300	2500
*12×17	12 "	6×14	2300	2600	2800
12×19	12 "	6×14	2300	2600	2800
*14×18	14 "	6×16	2800	3000	3200
*14×20	14 "	6×16	2900	3000	3200
*14×22	14 "	8×16	3000	3200	3400
*16×20	16 "	8×18	3600	4000	4250
*16×24	16 "	8×18	3700	4000	4250
*20×24	18 "	10×20	4800	5400	5750
*20×26	20 "	10×24	6000	7000	7450

This table gives different sizes of hot-air registers used in furnace practice, together with the equivalents of the capacity of the same in round leader pipes from furnace, with elevation of at least one inch to the foot; also equivalent in riser pipes (or stacks), and also the cubic feet of space on first, second, and third floors which said registers with their proper round and square pipes will heat. The table is based on normal conditions, with runs of pipe of usual length, and is intended to show the size of registers and pipes necessary to raise the temperature of air from zero outside to 70° on the inside, within reasonable time, without forcing. The sizes that are marked with an asterisk are those recommended for general use. The larger the register the less resistance to the flow of the heated air, but sizes mentioned will produce good results, and, being stock sizes, will always be found in stock. In planning work arrange to use the sizes referred to.

It should always be borne in mind, however, that uniform heating does not depend so much upon the *actual* size of the pipes as upon the *relative* sizes. For example, in a two-story house of eight rooms of *exactly the same size* and the same amount of wall and glass area the best heating results will be obtained not by using the same size of pipes for all the rooms, even if the pipes are of ample capacity, but by carefully proportioning the sizes of the pipes according to the exposure, length of the leaders, and whether the room is in the first or second story. The registers in the rooms with north and west exposures should be a little nearer the furnace, if possible, than the others, and the pipes to the first story should be larger than those leading to the second story.

The International Heater Company states that 1 sq. in. of capacity of hot-air pipe will heat 50 cu. ft. in stores and 90 cu. ft. in churches when there is but one pipe directly over the furnace.

Cold-air Box.—The sectional area of the cold-air box should be equal to three-fourths of the aggregate sectional area of the leaders. The box, or duct, should be 10 or 12 ins. deep (for dwellings) and wide enough to give the required sectional area. It should also always be provided with a damper, so that the supply may be regulated to the heavy winds and extreme cold weather.

Specifications.

The following form is given as a guide to architects in preparing the specifications for furnace work:

SPECIFICATIONS FOR FURNACE WORK IN RESIDENCE FOR MR. ....TO BE BUILT AT..................

................*Architect.*

Furnace.—Furnish and set up complete, where shown on basement plan, one No. — —— furnace, portable pattern, with double casings. Connect the furnace with the chimney with No. 24 galvanized-iron smoke-pipe of the same size as the collar on the furnace; all bends or turns to be made with three-piece elbows; the pipe to be strongly supported by wire, and to be kept 12 ins. below the ceiling.

Air-pit.—Excavate for and build a cold-air chamber under the furnace not less than 18 ins. deep, with 8-inch brick walls, laid and plastered with cement; also cement the bottom of the chamber. Build the cold-air duct under cellar floor, where shown on plan, to be — ft. long, 14 ins. deep in the clear, and — ins. wide, with sides of hard brick in cement, and the sides and bottom smoothly plastered with cement. Cover the duct with 3-inch flag-stones with tight joints, leaving opening of proper size for the wooden box to be built by the carpenter (wooden box should be included in carpenter's specifications).

Hot-air Pipes.—Furnish and properly connect with furnace and register boxes, leaders and stacks of the following sizes, all to be made of bright IX tin, and the stacks to be double with air space between. All turns in leaders to be made by three- or four-piece elbows, and the stacks to have boots or starters of approved pattern.

SIZES OF PIPES AND REGISTERS.

Hall............	12″	leader	No stack		12″×15″	register
Parlor..........	10″	"	4″×14″	stack	10″×12″	"
Dining-room....	12″	"	6″×12″	"	12″×15″	"
Library.........	10″	"	4″×14″	"	10″×12″	"
Chamber No. 1...	9″	"	4″×14″	"	9″×14″	"
" " 2...	9″	"	4″×12″	"	9″×12″	"
" " 3...	8″	"	4″×10″	"	8″×10″	"

Registers.—All registers are to be of sizes given in the foregoing list, of the Tuttle and Bailey manufacture, japanned,

except those in the first story, which are to be electro-bronze-plated. All floor registers are to be set in iron borders corresponding with the registers.

Register Boxes.—All register boxes to be made double; for first-floor boxes the *joists are to be lined with tin* and provided with *ceiling plates* full size of register, with plaster collar attached, so that pipes and boxes can be removed without disturbing the plastering or defacing the ceiling.

Miscellaneous.—All horizontal pipes in the basement to be round, and where they pass through partitions they are to be provided with collars, so that the pipes can be removed without disturbing the plastering. All leaders to be provided with dampers and tin tags designating the different rooms they supply; and whenever pipes run near woodwork the same is to be properly covered with tin and protected from any danger from fire. The contractor is to remove all rubbish made by him, clean up all ironwork, and leave the whole apparatus in complete working order, and furnish a poker of proper size.

Guarantee.—The contractor is to guarantee that the furnace shall, under proper management, heat all rooms with registers connected with the furnace to 70° Fahr. when temperature outside indicates 10° below zero. In event of the failure of the furnace to do this, the contractor is either to make the furnace heat said rooms or substitute another furnace that will heat the rooms at his own expense and without unnecessary delay.

Hot Air and Water Combination.

It is quite difficult, if not impossible, to heat throughout dwellings covering more than 1,400 sq. ft. with warm air alone. On account of the much larger exposure and the increased length of leaders, it becomes necessary to supplement the warm air with an auxiliary heat which can be carried to remote and exposed parts of the house, and which will not be affected by pressure of wind or long and crooked pipes. For supplying this auxiliary heat, hot water has been found best adapted as a rule, and a great variety of "combination" furnaces are now made which contain provisions for heating water which may be carried by pipes to radiators located in the portions of the house most difficult to heat by warm air. Such combination systems have been used with great success, and for heating

dwellings of ten average-size rooms the author believes it to be the most successful system, as it guarantees the comfortable warming of the house, and, if properly put in, thorough ventilation, which cannot be obtained by any system of direct hot-water or steam radiation. It is claimed that nearly 200 sq. ft. of hot-water radiation can be obtained by absorbing the surplus heat which would usually be wasted in a warm-air furnace.

The construction of the parts for heating the water varies greatly with different makes of furnaces. Some furnaces have a portion of the fire-pot hollow, and the water is heated there; others have a separate heater suspended over the fire-pot. It is impossible here to consider the relative merits of the various heaters; the architect should examine the heaters for himself and look up their record before specifying any particular make.

As a rule, the portions of the house which should be heated by the hot water are the halls, bathroom, and perhaps the rooms on the north or west side of the house.

The same rules govern the size of the radiators and piping and the manner of installing as in an entire hot-water plant.

Hot Air and Steam Combination.—There are also several furnaces which have a small steam-boiler placed above the fire by means of which a few rooms may be heated by direct steam radiation. Safety-valves are provided so that the steam pressure cannot exceed 5 lbs., and if the directions for running the apparatus are followed, the apparatus is perfectly safe. The steam combination possesses some advantages over the hot-water combination, and for a large residence the author believes that it will give more satisfactory results with intelligent management.

Hot-water Heating in Residences.—As stated on p. 1173, there is no better system of warming residences of ten or twelve rooms than the hot-water system, and it is being used to a greater extent every year.

The general principles of hot-water heating, as explained on pp. 1168 to 1172, apply to residences as to all other buildings. The open-tank system should always be used for this class of work. The following *Advice to Fitters*, published by the Gurney Heater Manufacturing Company, contains many practical suggestions that should be of almost equal interest to the architect and owner:

"When estimating upon a job, take well into consideration the extent of all flow, return pipes, and risers, also their situation,

and calculate them as radiating surface in addition to what is placed in rooms, and allow *heater* power accordingly.

"Due care must be exercised to provide for any special conditions, such as exposure of building, material of construction, location, length and size of mains governing plant under consideration.

"Allowances should be made for loose construction of doors and windows, which admit large volumes of cold air, and if there are outside doors which are used frequently and open directly into the room, a radiator should be placed near them.

"In estimating the radiating surface, it should be borne in mind that a large surface at a comparatively low temperature gives a much pleasanter atmosphere than a small surface at a high temperature.

"Excess of surface is no discomfort, as is the case with steam, since the temperature can easily be controlled by varying the fire or by valve on radiator.

"All flow and return pipes in cellar should be properly covered with hair-felt or some other good non-conducting material, to obtain the best and most economical results. *Doing this will save one sixth of the heat.* If no covering is used, paint all exposed pipes in basement a black or maroon japan. The heater should be neatly plastered with plastic asbestos."

Indirect Radiation.—Every large residence heated, either by hot-water or steam radiation, should have at least two indirect radiators, to provide for some ventilation. These should be placed in the cellar and connected with registers in the front hall and principal living-room. The common method of providing for indirect radiation is explained on pp. 1129 and 1130.

Direct radiation, as has been explained elsewhere, simply heats the air in the room over and over, and not only does *not* afford any ventilation, but tends to decrease the vitalizing qualities of the air.

Specification.

The following may serve as a guide in specifying hot-water heating for residences:

SPECIFICATION FOR HOT-WATER HEATING APPARATUS IN RESIDENCE FOR JOHN JONES, ESQ., BROOKLINE, MASS.

This specification contemplates a complete two-pipe circulating system, guaranteed perfect in every respect.

Heater.—Furnish and set up in cellar where shown on plan one No. (55 Ideal portable) water-boiler, guaranteed free from all flaws and defects.

The heater to set on a substantial foundation of hard brick laid in cement mortar and put in by the heating contractor.

Furnish and deliver one set of fire tools, consisting of one poker, one slice-bar, and one fine brush and handle.

Smoke-pipe.—Connect the boiler to the chimney by means of smoke-pipe made of No. 20 galvanized iron, the diameter of the pipe to be equal to the outlet on the heater.

Trimmings.—The boiler to be provided with one expansion thermometer registering from 80° F. to 250° F. Attach to main flow pipe, near the boiler, one Standard altitude gauge.*

Water Connections and Blow-off.—Feed-water with its supply pipe will be brought within 6 ft. of the boiler by the plumber and left with one $\frac{3}{4}$-inch cast-iron fitting for boiler connection, which is to be made by this contractor, with suitable cock.

Draw-off cock to be placed on lowest point of system and to be fitted for hose-nipple attachment.

Pipes.—Furnish and run all necessary flow and return pipes of ample size, connecting them to radiators with pipes of ample size to insure the free and rapid flow of hot water to the radiators and easy flow of the cooler water back to the heater.

All connections from risers to radiators to be made below floors.

Quality of Materials.—All materials used in the construction of this apparatus are to be the best of their respective kinds, all fittings to be heavily beaded and made of the best gray iron with clean-cut threads, and, when practicable, Y's and 45° L's are to be used.

Reaming.—The ends of all pipes used in the construction of this apparatus are to be reamed out and all obstructions removed before pipes are placed in position.

All flow and return pipes in basement to be supported by neat, strong, adjustable hangers, arranged to suit expansion and contraction, and properly secured to timbers overhead.

At all points where pipes pass through ceilings, floors, or partitions, the pipes shall be encased in iron or tin tubes and the holes protected with floor or ceiling plates.

Expansion Tank.—The expansion tank to be made of No. 22 galvanized iron, 30 ins. high and 14 ins. in diameter, and is to be furnished with a proper gauge-glass with brass mountings complete. It is to be placed above all the radiators in some suitable place and supported on a proper shelf. From this tank an overflow pipe will be run to basement or other suitable place with a vent pipe through the roof.

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* An altitude gauge indicates the amount of water in the system and is a convenient attachment which avoids the necessity of consulting the gauge-glass in the tank. It can be dispensed with if desired.

Radiators.—Furnish and set up the following radiators, viz.:

	No. of Radiators.	Square Feet of Radiating Surface.
Main hall.	1 indirect radiator	108 sq. ft.
Sitting-room.	1 " "	120 " "
Library.	1 direct radiator	40 " "
Dining-room.	1 " "	60 " "
Sitting-room chamber.	1 " "	40 " "
Library chamber.	1 " "	44 " "
Dining-room chamber.	1 " "	36 " "
Kitchen chamber.	1 " "	32 " "
Bathroom.	1 " "	32 " "
	9 radiators	512 sq. ft.

In all 284 sq. ft. of direct surface and 228 sq. ft. of indirect; total surface, 512 sq. ft.

The direct radiators to be (American Radiator Co.'s Rococo hot-water pattern) 38 ins. high.

Air-valves.—Each radiator will have properly connected to it a nickel-plated air-valve to be opened and closed with a key.

Radiator Valves.—All direct radiators will be promptly connected to the system of piping with a (*Gurney*) quick-opening nickel-plated radiator valve and union elbow.

Indirect Radiation.—The indirect radiators shall consist of two stacks of the (American Radiator Co.'s Excelsior hot-water radiator) connected together with tight joints and firmly suspended from the basement ceiling by suitable wrought-iron hangers.

The stacks shall be so piped and hung as to permit a quick, noiseless, and constant flow throughout of the heated water.

Each stack to be enclosed in galvanized iron chamber with proper inlet for fresh air and a corresponding outlet for warm air, connected by a galvanized pipe to the register in the room which the stack is intended to heat.

The registers to be of the Tuttle & Bailey pattern, electro bronzed plated, and of the following sizes: hall, 12×19; sitting-room, 14×22.

To have floor borders and to be set in a register box. The pipe connecting the stack and register is to be so arranged that all fresh air coming in will be properly heated and conveyed without loss to its destination. In arranging indirect boxes, care is to be exercised in getting ample space for cold air under the stack, and a corresponding space for warm air over the stack; unless otherwise specified, this space is not to be less than 12 ins. above and 10 ins. below the stack.

Covering of Pipe.—All flow and return pipe and fittings in cellar above the floor to be properly covered with 1-inch hair-

felt neatly sewed up in canvas and painted one coat of good white lead, or to be covered with asbestos or magnesia sectional covering with canvas cover and secured by brass lacquered bands.

Boiler Covering.—Cover all exposed parts of boiler, except the front, with plastic asbestos ($1\frac{1}{2}$) inches thick, neatly applied and trowelled smooth.

Workmanship.—All work to be done in a neat, substantial, and workmanlike manner, and the apparatus, when completed, to be thoroughly tested and left in good working order.

Guarantee.—The contractor is to guarantee that the apparatus, when completed in accordance with this specification, will be of ample capacity to evenly maintain a temperature of 70° F. in the rooms in which radiators are located when the outside temperature is at zero, and that the apparatus throughout will have a free and rapid circulation when in operation.

Steam Heating for Residences.

Although hot water is perhaps more popular just now for residence heating, there can be no question that a building can be as thoroughly warmed and ventilated by steam as by any other system, and generally at a smaller first cost. In very cold weather, it is doubtful if hot-water heating is as satisfactory as steam.

For indirect radiation, steam heat is generally considered cheaper than hot-water heat, and in every way as satisfactory.

For very large residences, the author would recommend steam heat, all of the principal rooms to be heated by indirect radiation, and only the bathroom, halls, and perhaps the attic and one or two rooms on the north side, which generally includes the dining-room, by direct radiation. For dining-rooms a special direct radiator, containing a warming closet, is made.

The air-supply to the indirect stacks should be very large and provided with a damper, so that the supply may be regulated according to the weather.

The same principles apply in heating a residence by steam as in heating any other building, and there is no difference in the piping and radiators. The boilers used in residence heating, however, are generally of the cast-iron sectional type described on pp. 1141 to 1144.

The single-pipe system is commonly used in dwellings, all indirect radiators, however, being connected with a return pipe dropped below the water-line. Two specifications are appended for steam heating, one for all direct radiation and one for all indirect radiation; the latter can be easily amended to provide for some direct radiation.

Typical Specification.

For a First-class Low-pressure Steam-heating Apparatus for Heating by All Direct Radiation.

Intention.—This specification is intended to cover everything necessary to fully finish and install in the above-mentioned building a complete steam-heating system in strict accordance with the plans and this specification, as prepared by T. Square, architect.

Plans.—The plans herewith are intended to show only the location of the boiler, piping, and radiators; the arrangement of the piping will be left largely to the contractor, subject to the approval of the architect.

General Requirement.—This contractor is to provide all necessary tools and appliances for the erection and completion of the work, and when completed must remove all apparatus, refuse, and débris from the building and grounds, leaving the work in a clean, uninjured, and perfect condition. No cutting of any description tending to weaken the building structurally shall be undertaken without consulting the architects.

This contractor shall be fully responsible for the safety and good condition of the work and material embraced in this contract until the completion and acceptance of the same.

All work must be of the best quality, and should at any time improper, imperfect, or unsound material or faulty workmanship be observed, whether before or after same has been built into the structure, this contractor shall, upon notice from the architect, remove same and good and proper material and workmanship be substituted without delay in place thereof, in default of which the architect will effect same by other means as may be deemed best, and shall deduct the cost of such alterations from the sum due the contractor under this contract.

System.—The heating to be effected by direct radiation distributed throughout as shown on plans, and the circulation of the steam shall be by the one-pipe basement system.

Boiler.—This contractor is to build foundation for boiler, where shown, 12″ deep, of common hard brick laid in cement mortar. Leave ash-pit for boiler of proper size, 12″ deep, cemented, and made water-tight. Furnish and set up one (— Ideal C. I. sectional) boiler, provided with 6″ low-pressure brass-cased steam-gauge, water-gauge, and glass, gauge-cocks, ombination column, safety- and blow-off valves, and all other usual and necessary trimmings to complete the boiler,* and full set of fire tools, consisting of one slicing-bar, one hoe, one poker, and a cleaning brush. Cover boiler with $1\frac{1}{4}$″ of asbestos cement, neatly trowelled to a smooth finish.

Water Feed.—The plumber will bring the water-supply to

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* For house-heating plants it is well to specify also "one automatic damper regulator of approved pattern, with connection for operating draught door and cold-air check."

within 6 ft. of boiler, and this contractor is to make connection with boiler with $\frac{3}{4}''$ iron pipe, stop-cock, and check-valve.

Catch-basin.—Furnish and set one cast-iron catch-basin, $28'' \times 36''$, where shown on plans. Connect the boiler blow-off pipe to catch-basin, connect same to sewer, and vent with $1\frac{1}{4}''$ pipe to roof. (Note. Catch-basins are usually omitted in house heating, a hose being attached to the blow-off valve for blowing off.)

Smoke-pipe.—Connect the boiler with the chimney with a round smoke-pipe made of No. 16 black iron with suitable balance damper. This connection to be of same size as left for this purpose by maker of boiler.

Main Pipes and Risers.—The main steam-pipe is to be of ample size to carry all the risers and radiators attached to the system, and is to be so graded that all water of condensation will flow freely back to the boiler without noise. From the top of this main the various branches are to be taken to radiators and risers, the connections for which are to be so made that no traps are formed, and when horizontal runs occur they are to have a relief pipe to carry off all water of condensation.

Eccentric fittings only to be used on heating mains where reduced in size.

Sizes of supplies to radiators to be $1''$ to 24 sq. ft., $1\frac{1}{4}''$ to 60 sq. ft., and $1\frac{1}{2}''$ to all above. Radiators on first floor to be connected direct to steam main.

All horizontal pipes to be one size larger than the vertical pipes.

All steam connections from heating main to radiators and risers to run on a 45° angle from heating main and to be one size larger than risers and radiator feeds.

Pipe and Fittings.—All pipe used throughout shall be of the best quality wrought-iron pipe of standard weight and thickness, smooth inside, free from imperfections, and true to shape. All threads to be clean-cut, straight, and true. All fittings to be of the best heavy gray iron, with taper threads and heavy beaded. No inferior pipe or fittings will be allowed.

All couplings used must be of the best make, with recessed ends, except reducers, which are to be offset.

Supports.—All piping to be supported by approved expansion hangers or rollers not to exceed 10 ft. apart. Use neat cast-iron floor and ceiling plates where pipes pass through floors, ceilings, and partitions.

Radiators.—Furnish direct radiation to the amount as enumerated on plans of the American Radiator Co.'s make or equal, all $38''$ radiators to be the (Perfection) pattern. All radiators $14''$ high to be "Ætna flue."

Radiator Valves.—The radiators are to be furnished with Jenkins disc union valves of the best metal nickel-plated and have hard-wood handles.

Valves.—All valves $2''$ and under to have brass bodies and iron wheels, over $2''$ to be heavy cast iron with brass stem and

trimmings and iron wheels; all valves 4″ and over to have heavy yokes. All gate-valves to be of (Crane Co.'s) make or equal.

Air-valves.—Each radiator throughout the entire building shall be furnished with a Marsh automatic air-valve.

Painting and Bronzing.—All radiators and exposed pipes in rooms or halls are to be neatly painted two coats best radiator enamel or bronzed in desired colors.

Finally.—When completed, test the apparatus to 20 lbs. pressure and make tight at that pressure, said test to be conducted under the supervision of the architect. Fuel for the test to be furnished by the owner, and when accepted the apparatus is to be turned over to the owner in complete working order.

All valves and stuffing-boxes to be properly packed and the plant to be completed in all its parts, it being understood that this contractor is to furnish all miscellaneous material, tools, labor, etc., necessary to complete the work in a first-class and workmanlike manner.

Guarantee.—This contractor shall guarantee that when the apparatus is completed in accordance with these specifications and drawings it will be free from all mechanical defects and of ample capacity to heat all rooms where radiation is placed to a temperature of 70° when the outside temperature is 10° below zero.

Typical Specification.

For a Superior Low-pressure Steam-heating Apparatus for Heating by the Indirect System, with a Steam Pressure of from One to Five Pounds per Square Inch.

[Note.—This specification should be accompanied by a heating plan showing the position of all indirect radiation stacks in the basement, the cold-air inlets to the same, and the size and general position of the main steam and return pipes and the connections to the radiators.

The return main should be placed at or below the floor and the steam main close under the ceiling.

The size and location of warm-air registers should be shown on the general floor plans.]

General Requirements.—Boiler, trimmings, and smoke and feed connection same as in preceding specification.

System of Piping.—The system of piping throughout will be constructed on the double-pipe "gravity return" plan, and all pipes erected will be of ample size to insure the active delivery of dry steam to the radiators and easy flow of the water of condensation back to the boiler.

Furnish and erect all supply and steam mains and branch connecting pipes of the sizes and located in the relative positions shown on the plans. All piping to be graded and properly dripped and the steam mains to be hung in position by means of expansion pipe-hangers.

The main return to be run at or under basement floor and protected from moisture.*

Pipe Covering.—All pipes in the cellar above the floor to be covered with asbestos (or magnesia) sectional covering with canvas cover and secured by brass lacquered bands.

Stack Boxes and Flues.—The heating of the several apartments will be accomplished by means of indirect radiators set in clusters of "stacks," each hung from the ceiling of the cellar, and the heat from these "stacks" will be conveyed to the room to be heated by means of tin hot-air pipes set in the walls and leading from stack to the room to be heated; each room heated to have an independent "stack" and to be connected therewith by an independent tin hot-air pipe. Each of the "stacks" of indirect radiators to be enclosed in a well-made box or galvanizd iron chamber and from each "stack" a galvanized-iron duct of proper size is to lead to the opening in outside wall provided for the same. Place a damper in the cold-air duct and a tight door in the stack casing below the radiator.

The radiators shall be hung and the chambers made so that there shall be a space of not less than (12) ins. above and (10) ins. below the stack, and the cold-air duct shall connect with the bottom of the chamber, at a point farthest from the warm-air outlet.

Radiators.—Furnish and erect in cellar, in the positions as shown on plans, ten "stacks" of approved pattern indirect radiators that in the aggregate will contain not less than (732) sq. ft. of radiating surface, and divided up for the several rooms to be heated as follows, viz.:

First Story:							
Hall	1	"stack"	to	contain	108	sq.	ft.
Parlor	1	"	"	"	96	"	"
Dining-room	1	"	"	"	108	"	"
Library	1	"	"	"	96	"	"
Rear hall	1	"	"	"	48	"	"
Second Story:							
Chamber over parlor	1	"stack"	to	contain	72	sq.	st.
Chamber over dining-room	1	"	"	"	72	"	"
Chamber over library	1	"	"	"	72	"	"
Hall bedroom	1	"	"	"	36	"	"
Bathroom	1	"	"	"	24	"	"

No valves are to be placed in either the supply or return to radiating stack, but an improved automatic air-valve must be placed on each stack.

Pipe Covering.—All cellar pipes to be neatly covered with asbestos sheathing, then 1-inch thick hair-felt and canvas casing sewed on.

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* In damp situations the return pipes, when necessary to drop below floor, should be run in brick ducts laid in cement mortar and the pipe packed with mineral wool or asbestos.

Registers.—Furnish and set in position in each room heated a vertical wheel register of the size shown on plans. All registers for first story to be bronze finish, and all others to be black or white japanned finish, as shall be selected.

Tin and Galvanized-iron Work.—Furnish to builder (and by him to be set in position as shown on plans) all tin wall pipes and register boxes for hot air to the rooms to be heated, all to be made of IX tin and of the sizes shown on plan.

Furnish and erect the galvanized-iron casings for the ten stacks as above specified, with galvanized-iron ducts to the outside openings, to be constructed in a substantial and workmanlike manner.

Guarantee.—The contractor is to guarantee that the apparatus when completed will be of ample capacity to maintain an even temperature of 70° F. in the rooms heated when the outside temperature is zero, and that the apparatus will afford free circulation throughout and be noiseless in operation.

Books on Residence Heating.—Much valuable information on residence heating may be obtained from pamphlets published by different manufacturers, among whom are the American Radiator Co., the International Heater Co., the Gurney Heater Manufacturing Co., Gorton & Lidgerwood Co., Isaac A. Sheppard & Co., and the Excelsior Steel Furnace Co., of Chicago. The latter company publish a very complete book on furnace heating and furnace fittings which every architect should have.

TABLES.

The following tables will be found useful in estimating the size of registers, piping, and heating surface of pipes and boiler tubes:

TABLE OF SIZES AND DIMENSIONS OF SAFETY DOUBLE HOT-AIR STACKS.

Made by the Excelsior Steel Furnace Company.

Size of Stack as Listed, in Inches.	Actual Size of Outside Stack.	Actual Size of Inside Stack.	Area of Inside Stack, in Inches.	Capacity as Compared with that of Hot-air Pipe with Pitch of 1 Inch to 1 Foot.	Equivalent in Round Pipe with Pitch of 1 Inch to 1 Foot.	Sizes of Round Pipe which should be Used with Each Stack.	Area of Said Round Pipes, in Inches.	Size of Registers and Register-boxes which Should be Used with Each Stack.	Cubic Feet of Space (approximate) that can be Heated with Each Stack with Pipe and Registers of Size Given.	Equivalent of Said Space on Floor of Rooms 10 Feet High.	Area, in Inches, of Registers, with Space Occupied by Bars Deducted.
4×8	3⅝×7⅝	3¼×7	23	35	6½	7	38	6×8	500	6×8	35
4×10	3⅝×9⅝	3¼×9	29	43	7½	8	50	8×10	850	8×10	45
4×11	3⅝×10⅝	3¼×10	32½	48	8	8	50	8×12	1000	9×11	55
4×12	3⅝×11⅝	3¼×11	35	53	8¼	9	63	9×12	1250	10×12½	60
4×14	3⅝×13⅝	3¼×13	41	63	9	9	63	10×12	1650	12×14	70
6×10	5⅜×9⅝	5¼×9	47	71	10	10	78	10×14	2000	12×17	80
6×12	5⅜×11⅝	5¼×11	58	87	11	12	113	12×15	2300	14×17	115
6×14	5⅜×13⅝	5¼×13	68	102	12	12	113	12×17	2600	15×18	120
6×16	5⅜×15⅝	5¼×15	79	119	12½	14	154	14×20	3000	15×20	156
8×18	7⅜×17⅝	7¼×17	124	186	15	16	201	16×24	4000	20×20	210
10×20	9⅜×19⅝	9¼×19	176	264	18	18	254	20×24	5400	20×27	270
10×24	9¾×23⅝	9¼×23	213	330	20½	20	314	21×29	7000	20×35	340

DIMENSIONS OF REGISTERS AND BORDERS.*

Made by the Tuttle & Bailey Manufacturing Co.

Size of Body.	Register.		Border.	
	Extreme Dimensions.	Depth Open.	With Ribs. Floor Opening.	Tin-box Size.
4×6	$5\frac{5}{8} \times 7\frac{5}{8}$	$1\frac{5}{8}$		
4×8	$5\frac{1}{4} \times 9\frac{1}{4}$	$2\frac{1}{4}$		
4×10	$5\frac{1}{4} \times 11\frac{1}{4}$	$2\frac{1}{4}$		
4×13	$5\frac{1}{4} \times 14\frac{1}{4}$	$2\frac{1}{4}$		
4×15	$5\frac{1}{4} \times 16\frac{3}{16}$	$2\frac{1}{4}$		
4×18	$5\frac{1}{4} \times 19\frac{1}{4}$	$2\frac{1}{4}$		
5×8	$6\frac{3}{8} \times 9\frac{3}{8}$	2	$8\frac{1}{8} \times 11\frac{1}{8}$	$5\frac{5}{16} \times 8\frac{5}{16}$
5×11	$6\frac{3}{8} \times 12\frac{3}{8}$	2	$8\frac{1}{8} \times 14\frac{1}{8}$	$5\frac{5}{16} \times 11\frac{5}{16}$
5×13	$6\frac{3}{8} \times 14\frac{3}{8}$	2	$8\frac{1}{8} \times 16\frac{1}{8}$	$5\frac{5}{16} \times 13\frac{5}{16}$
5×16	$6\frac{3}{8} \times 17\frac{3}{8}$	2	$8\frac{1}{8} \times 19\frac{1}{8}$	$5\frac{5}{16} \times 16\frac{5}{16}$
6×6	$7\frac{11}{16} \times 7\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 9\frac{9}{16}$	$6\frac{9}{16} \times 6\frac{9}{16}$
6×8	$7\frac{11}{16} \times 9\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 11\frac{9}{16}$	$6\frac{9}{16} \times 8\frac{9}{16}$
6×9	$7\frac{11}{16} \times 10\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 12\frac{9}{16}$	$6\frac{9}{16} \times 9\frac{9}{16}$
6×10	$7\frac{11}{16} \times 11\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 13\frac{9}{16}$	$6\frac{9}{16} \times 10\frac{9}{16}$
6×14	$7\frac{11}{16} \times 15\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 17\frac{9}{16}$	$6\frac{9}{16} \times 14\frac{9}{16}$
6×16	$7\frac{11}{16} \times 17\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 19\frac{9}{16}$	$6\frac{9}{16} \times 16\frac{9}{16}$
6×18	$7\frac{11}{16} \times 19\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 21\frac{9}{16}$	$6\frac{9}{16} \times 18\frac{9}{16}$
6×24	$7\frac{11}{16} \times 25\frac{11}{16}$	$2\frac{3}{8}$	$9\frac{9}{16} \times 27\frac{9}{16}$	$6\frac{9}{16} \times 24\frac{9}{16}$
7×7	$8\frac{11}{16} \times 8\frac{11}{16}$	$2\frac{3}{4}$	$10\frac{9}{16} \times 10\frac{9}{16}$	$7\frac{9}{16} \times 7\frac{9}{16}$
7×10	$8\frac{11}{16} \times 11\frac{11}{16}$	$2\frac{3}{4}$	$10\frac{9}{16} \times 13\frac{9}{16}$	$7\frac{9}{16} \times 10\frac{9}{16}$
8×8†	$9\frac{3}{4} \times 9\frac{3}{4}$	3	$11\frac{7}{8} \times 11\frac{7}{8}$	$8\frac{5}{8} \times 8\frac{5}{8}$
8×10	$9\frac{3}{4} \times 11\frac{3}{4}$	3	$11\frac{7}{8} \times 13\frac{7}{8}$	$8\frac{5}{8} \times 10\frac{5}{8}$
8×12†	$9\frac{3}{4} \times 13\frac{3}{4}$	3	$11\frac{7}{8} \times 15\frac{7}{8}$	$8\frac{5}{8} \times 12\frac{5}{8}$
8×15	$9\frac{3}{4} \times 16\frac{11}{16}$	3	$11\frac{7}{8} \times 18\frac{7}{8}$	$8\frac{5}{8} \times 15\frac{5}{8}$
8×18	$9\frac{3}{4} \times 19\frac{3}{4}$	3	$11\frac{7}{8} \times 21\frac{7}{8}$	$8\frac{5}{8} \times 18\frac{5}{8}$
8×21	$9\frac{3}{4} \times 22\frac{3}{4}$	3	$11\frac{7}{8} \times 24\frac{7}{8}$	$8\frac{5}{8} \times 21\frac{5}{8}$
8×24	$9\frac{3}{4} \times 25\frac{3}{4}$	3	$11\frac{7}{8} \times 27\frac{7}{8}$	$8\frac{5}{8} \times 24\frac{5}{8}$
9×9	$10\frac{7}{8} \times 10\frac{7}{8}$	$3\frac{1}{4}$	$13\frac{1}{16} \times 13\frac{1}{16}$	$9\frac{11}{16} \times 9\frac{11}{16}$
9×12†	$10\frac{7}{8} \times 13\frac{7}{8}$	$3\frac{1}{4}$	$13\frac{1}{16} \times 16\frac{1}{16}$	$9\frac{11}{16} \times 12\frac{11}{16}$
9×13	11×15	$3\frac{1}{4}$	$13\frac{1}{16} \times 17\frac{1}{16}$	$9\frac{11}{16} \times 13\frac{11}{16}$
9×14†	$10\frac{7}{8} \times 15\frac{7}{8}$	$3\frac{1}{4}$	$13\frac{1}{16} \times 18\frac{1}{16}$	$9\frac{11}{16} \times 14\frac{11}{16}$
9×16	$10\frac{7}{8} \times 17\frac{13}{16}$	$3\frac{1}{4}$	$13\frac{1}{16} \times 20\frac{1}{16}$	$9\frac{11}{16} \times 16\frac{11}{16}$
9×18	$10\frac{7}{8} \times 19\frac{7}{8}$	$3\frac{1}{4}$	$13\frac{1}{16} \times 22\frac{1}{16}$	$9\frac{11}{16} \times 18\frac{11}{16}$
9×20	$10\frac{7}{8} \times 21\frac{7}{8}$	$3\frac{1}{4}$	$13\frac{1}{16} \times 24\frac{1}{16}$	$9\frac{11}{16} \times 20\frac{11}{16}$
10×10	$11\frac{15}{16} \times 11\frac{15}{16}$	$3\frac{3}{8}$	$14\frac{3}{16} \times 14\frac{3}{16}$	$10\frac{11}{16} \times 10\frac{11}{16}$
10×12	$11\frac{15}{16} \times 13\frac{15}{16}$	$3\frac{3}{8}$	$14\frac{3}{16} \times 16\frac{3}{16}$	$10\frac{11}{16} \times 12\frac{11}{16}$
10×14	$12 \times 15\frac{15}{16}$	$3\frac{3}{8}$	$14\frac{3}{16} \times 18\frac{3}{16}$	$10\frac{11}{16} \times 14\frac{11}{16}$
10×16	$11\frac{15}{16} \times 17\frac{7}{8}$	$3\frac{3}{8}$	$14\frac{3}{16} \times 20\frac{3}{16}$	$10\frac{11}{16} \times 16\frac{11}{16}$
10×18	$11\frac{15}{16} \times 19\frac{7}{8}$	$3\frac{3}{8}$	$14\frac{3}{16} \times 22\frac{3}{16}$	$10\frac{11}{16} \times 18\frac{11}{16}$
10×20	$11\frac{15}{16} \times 21\frac{7}{8}$	$3\frac{3}{8}$	$14\frac{3}{16} \times 24\frac{3}{16}$	$10\frac{11}{16} \times 20\frac{11}{16}$
12×12	$14\frac{1}{16} \times 14\frac{1}{16}$	4	$16\frac{7}{16} \times 16\frac{7}{16}$	$12\frac{13}{16} \times 12\frac{13}{16}$
12×14	$14\frac{1}{16} \times 16\frac{1}{16}$	4	$16\frac{7}{16} \times 18\frac{7}{16}$	$12\frac{13}{16} \times 14\frac{13}{16}$
12×15	$13\frac{13}{16} \times 16\frac{15}{16}$	4	$16\frac{7}{16} \times 19\frac{7}{16}$	$12\frac{13}{16} \times 15\frac{13}{16}$

* For special side-wall registers, see p. 1201.

† These sizes are those most likely to be found in stock of local dealers.

DIMENSIONS OF REGISTERS AND BORDERS.—*Cont.*

Size of Body.	Register.		Border.	
	Extreme Dimensions.	Depth Open.	With Ribs. Floor Opening.	Tin-box Size.
12×16	$14\frac{1}{16}\times 18$	4	$16\frac{7}{16}\times 20\frac{7}{16}$	$12\frac{13}{16}\times 16\frac{13}{16}$
12×17*	$14\frac{1}{16}\times 19$	4	$16\frac{7}{16}\times 21\frac{7}{16}$	$12\frac{13}{16}\times 17\frac{13}{16}$
12×18	$14\frac{1}{16}\times 20\frac{1}{16}$	4	$16\frac{7}{16}\times 22\frac{7}{16}$	$12\frac{13}{16}\times 18\frac{13}{16}$
12×19	$14\frac{1}{16}\times 21\frac{1}{16}$	4	$16\frac{7}{16}\times 23\frac{7}{16}$	$12\frac{13}{16}\times 19\frac{13}{16}$
12×20	$14\frac{1}{16}\times 22$	4	$16\frac{7}{16}\times 24\frac{7}{16}$	$12\frac{13}{16}\times 20\frac{13}{16}$
12×24	$14\frac{1}{16}\times 26$	4	$16\frac{7}{16}\times 28\frac{7}{16}$	$12\frac{13}{16}\times 24\frac{13}{16}$
12×30	$14\frac{1}{16}\times 32$	4		
12×36	$14\frac{1}{16}\times 38$	4		
14×14	$16\frac{5}{16}\times 16\frac{5}{16}$	4	$18\frac{15}{16}\times 18\frac{15}{16}$	$14\frac{7}{8}\times 14\frac{7}{8}$
14×16	$16\frac{5}{16}\times 18\frac{5}{16}$	4	$18\frac{15}{16}\times 20\frac{15}{16}$	$14\frac{7}{8}\times 16\frac{7}{8}$
14×18	$16\frac{3}{8}\times 20\frac{5}{16}$	4	$18\frac{15}{16}\times 22\frac{15}{16}$	$14\frac{7}{8}\times 18\frac{7}{8}$
14×20	$16\frac{5}{16}\times 22\frac{5}{16}$	4	$18\frac{15}{16}\times 24\frac{15}{16}$	$14\frac{7}{8}\times 20\frac{7}{8}$
14×22	$16\frac{3}{8}\times 24\frac{1}{4}$	4	$18\frac{15}{16}\times 26\frac{15}{16}$	$14\frac{7}{8}\times 22\frac{7}{8}$
15×25	$17\frac{11}{16}\times 27\frac{11}{16}$	$4\frac{1}{2}$	$19\frac{13}{16}\times 29\frac{15}{16}$	$16\frac{1}{8}\times 26\frac{1}{4}$
16×16	$18\frac{5}{16}\times 18\frac{5}{16}$	$4\frac{1}{4}$	$20\frac{7}{8}\times 20\frac{7}{8}$	$16\frac{7}{8}\times 16\frac{7}{8}$
16×18	$18\frac{5}{16}\times 20\frac{5}{16}$	$4\frac{1}{4}$	$20\frac{7}{8}\times 22\frac{7}{8}$	$16\frac{7}{8}\times 18\frac{7}{8}$
16×20	$18\frac{1}{16}\times 22\frac{5}{16}$	$4\frac{1}{4}$	$20\frac{7}{8}\times 24\frac{7}{8}$	$16\frac{7}{8}\times 20\frac{7}{8}$
16×22	$18\frac{5}{16}\times 24\frac{5}{16}$	$4\frac{1}{4}$	$20\frac{7}{8}\times 26\frac{7}{8}$	$16\frac{7}{8}\times 22\frac{7}{8}$
16×24	$18\frac{5}{16}\times 26\frac{11}{16}$	$4\frac{1}{4}$	$20\frac{7}{8}\times 28\frac{7}{8}$	$16\frac{7}{8}\times 25\frac{1}{4}$
16×28	$18\frac{5}{16}\times 30\frac{5}{16}$	$4\frac{1}{4}$	$20\frac{7}{8}\times 32\frac{7}{8}$	$16\frac{7}{8}\times 28\frac{7}{8}$
16×32	$18\frac{5}{16}\times 34\frac{5}{16}$	$4\frac{1}{4}$	$20\frac{7}{8}\times 36\frac{7}{8}$	$16\frac{7}{8}\times 32\frac{7}{8}$
18×18	$20\frac{5}{16}\times 20\frac{5}{16}$	$4\frac{3}{4}$	$22\frac{15}{16}\times 22\frac{15}{16}$	$18\frac{7}{8}\times 18\frac{7}{8}$
18×21	$20\frac{5}{16}\times 23\frac{5}{16}$	$4\frac{3}{4}$	$22\frac{15}{16}\times 25\frac{15}{16}$	$18\frac{7}{8}\times 21\frac{7}{8}$
18×24	$20\frac{5}{16}\times 26\frac{5}{16}$	$4\frac{3}{4}$	$22\frac{15}{16}\times 28\frac{15}{16}$	$18\frac{7}{8}\times 24\frac{7}{8}$
18×27	$20\frac{5}{16}\times 29\frac{5}{16}$	$4\frac{3}{4}$	$22\frac{15}{16}\times 31\frac{15}{16}$	$18\frac{7}{8}\times 27\frac{7}{8}$
18×30	$20\frac{5}{16}\times 32\frac{1}{4}$	$4\frac{3}{4}$	$22\frac{15}{16}\times 34\frac{15}{16}$	$18\frac{7}{8}\times 30\frac{7}{8}$
18×36	$20\frac{5}{16}\times 38\frac{1}{4}$	$4\frac{3}{4}$	$22\frac{15}{16}\times 40\frac{15}{16}$	$18\frac{7}{8}\times 36\frac{7}{8}$
20×20	$22\frac{3}{8}\times 22\frac{3}{8}$	$5\frac{1}{2}$	$25\frac{1}{8}\times 25\frac{1}{8}$	$20\frac{15}{16}\times 20\frac{15}{16}$
20×24	$22\frac{3}{8}\times 26\frac{3}{8}$	$5\frac{1}{2}$	$25\frac{1}{8}\times 29\frac{1}{8}$	$20\frac{15}{16}\times 24\frac{15}{16}$
20×26*	$22\frac{9}{16}\times 28\frac{5}{8}$	$5\frac{1}{2}$	$25\frac{1}{8}\times 31\frac{1}{8}$	$20\frac{15}{16}\times 26\frac{15}{16}$
21×29	$23\frac{3}{8}\times 31\frac{3}{8}$	$5\frac{1}{2}$	$26\frac{1}{8}\times 34\frac{1}{8}$	$21\frac{15}{16}\times 29\frac{15}{16}$
24×24	$26\frac{7}{16}\times 26\frac{7}{16}$	$5\frac{3}{8}$	$29\frac{1}{2}\times 29\frac{1}{2}$	$24\frac{15}{16}\times 24\frac{15}{16}$
24×27	$26\frac{7}{16}\times 29\frac{3}{8}$	$5\frac{3}{8}$	$29\frac{1}{2}\times 32\frac{1}{2}$	$24\frac{15}{16}\times 27\frac{15}{16}$
24×30	$26\frac{7}{16}\times 32\frac{3}{8}$	$5\frac{3}{8}$	$29\frac{1}{2}\times 35\frac{1}{2}$	$24\frac{15}{16}\times 30\frac{15}{16}$
24×32	$26\frac{7}{16}\times 34\frac{3}{8}$	$5\frac{3}{8}$	$29\frac{1}{2}\times 37\frac{1}{2}$	$24\frac{15}{16}\times 32\frac{15}{16}$
24×36	$26\frac{7}{16}\times 38\frac{3}{8}$	$5\frac{3}{8}$	$29\frac{1}{2}\times 41\frac{1}{2}$	$24\frac{15}{16}\times 36\frac{15}{16}$
24×45	$26\frac{7}{16}\times 47\frac{3}{8}$	$5\frac{3}{8}$	$29\frac{1}{2}\times 50\frac{1}{2}$	$24\frac{15}{16}\times 45\frac{15}{16}$
27×27	$29\frac{7}{16}\times 29\frac{7}{16}$	6	$32\frac{1}{2}\times 32\frac{1}{2}$	$27\frac{15}{16}\times 27\frac{15}{16}$
27×38	$29\frac{7}{16}\times 40\frac{3}{8}$	$6\frac{1}{2}$	$32\frac{1}{2}\times 43\frac{1}{2}$	$27\frac{15}{16}\times 38\frac{15}{16}$
30×30	$32\frac{3}{8}\times 32\frac{3}{8}$	$7\frac{3}{4}$	$35\frac{1}{2}\times 35\frac{1}{2}$	$30\frac{15}{16}\times 30\frac{15}{16}$
30×36	$32\frac{3}{8}\times 38\frac{3}{8}$	$7\frac{3}{4}$	$35\frac{1}{2}\times 41\frac{1}{2}$	$30\frac{15}{16}\times 36\frac{15}{16}$
30×42	$32\frac{3}{8}\times 44\frac{3}{8}$	$7\frac{3}{4}$	$35\frac{1}{2}\times 47\frac{1}{2}$	$30\frac{15}{16}\times 42\frac{15}{16}$

* These sizes are those most likely to be found in stock of local dealers.

ESTIMATED CAPACITY OF PIPES AND REGISTERS.

ROUND PIPES.

Diameter of Pipe.	Area in Sq. Inches.	Diameter of Pipe.	Area in Sq. Inches.	Diameter of Pipe.	Area in Sq. Inches.
7 inches	38	12 inches	113	22 inches	380
8 "	50	14 "	154	24 "	452
9 "	63	16 "	201	26 "	531
10 "	78	18 "	254	28 "	616
11 "	95	20 "	314	30 "	707

RECTANGULAR PIPES.

Size of Pipe.	Area in Sq. Inches.	Size of Pipe.	Area in Sq. Inches.	Size of Pipe.	Area in Sq. Inches.
4×8	32	8×20	160	12×18	216
4×10	40	8×24	192	12×20	240
4×12	48	10×12	120	12×24	288
4×16	64	10×15	150	14×14	196
6×10	60	10×16	160	14×16	224
6×12	72	10×18	180	14×20	280
6×16	96	10×20	200	16×16	256
8×10	80	12×12	144	16×18	288
8×12	96	12×15	180	16×20	320
8×16	128	12×16	192	16×24	384

REGISTERS.

Size of Opening.	Capacity in Sq. Inches.	Size of Opening.	Capacity in Sq. Inches.	Size of Opening.	Capacity in Sq. Inches.
6×10	40	10×14	93	20×20	267
8×10	53	10×16	107	20×24	320
8×12	64	12×15	120	20×26	347
8×15	80	12×19	152	21×29	406
9×12	72	14×22	205	27×27	486
9×14	84	15×25	250	27×38	684
10×12	80	16×24	256	30×30	600

ROUND REGISTERS.

Size of Opening.	Capacity in Sq. Inches.	Size of Opening.	Capacity in Sq. Inches.	Size of Opening.	Capacity in Sq. Inches.
7 inches	26	12 inches	75	20 inches	209
8 "	33	14 "	103	24 "	301
9 "	42	16 "	134	30 "	471
10 "	52	18 "	169	36 "	679

T. & B. SPECIAL SIDE-WALL REGISTERS FOR SHALLOW FLUES AND THIN PARTITIONS FOR USE IN BASE-BOARDS.

This register has a single valve, and the front projects 2 inches into the room; with it a 6″×13″ flue can be used with 3″ studding, i.e., in the first-story rooms.

SIZES AND CAPACITY.

Register		Tin Flue.		Round Pipe.	
List Size.	Net Air Opening in Register Face.	Size.	Capacity.	Size.	Capacity.
Inches.	Sq. inches.	Inches.	Sq. inches.	Inches.	Sq. inches.
7×10	47	4 ×10	40	7	38
7×12	56	4 ×12	48	8	48
8×13	70	5 ×13	65	9	63
8×15	80	5 ×15	75	10	78
10×13	86	6 ×13	78	10	78
8×17	95	5½×17	93	11	95

DIAMETER OF MAIN AND BRANCH PIPES AND SQUARE FEET OF COIL SURFACE THEY WILL SUPPLY IN AN OPEN HOT-WATER APPARATUS WHEN COILS ARE AT DIFFERENT ALTITUDES FOR DIRECT RADIATION OR IN THE LOWER STORY FOR INDIRECT RADIATION.*

Diameter of Pipe, in Inches.	Indirect Radiation.	Direct Radiation. Height of Coil above Bottom of Boiler, in Feet.								
	0	10	20	30	40	50	60	70	80	100
	Sq.ft.	Sq.ft.	Sq.ft.	Sq. ft.	Sq. ft.	Sq. ft.	Sq. ft.	Sq. ft.	Sq. ft	Sq. ft.
¾	49	50	52	53	55	57	59	61	63	68
1	87	89	92	95	98	101	103	108	112	121
1¼	136	140	144	149	153	158	161	169	175	189
1½	196	202	209	214	222	228	235	243	252	271
2	349	359	370	380	393	405	413	433	449	483
2½	546	561	577	595	613	633	643	678	701	755
3	785	807	835	856	888	912	941	974	1,009	1,086
3½	1,069	1,099	1,132	1,166	1,202	1,241	1,283	1,327	1,374	1,480
4	1,395	1,436	1,478	1,520	1,571	1,621	1,654	1,733	1,795	1,933
4½	1,767	1,817	1,871	1,927	1,988	2,052	2,120	2,193	2,272	2,445
5	2.185	2,244	2,309	2,379	2,454	2,531	2,574	2,713	2,805	3,019
6	3,140	3,228	3,341	3,424	3,552	3,648	3,763	3,897	4,036	4,344
7	4,276	4,396	4,528	4,664	4,808	4,964	5,132	5,308	5,496	5,920
8	5,580	5,744	5,912	6,080	6,284	6,484	6,616	6,932	7,180	7,735
9	7,068	7,268	7,484	7,708	7,952	8,208	8,482	8,774	9,088	9,780
10	8,740	8,976	9,236	9,516	9,816	10,124	10,296	10,852	11,220	12,076

* F. Schumann, C.E.

DIAMETER OF STEAM-SUPPLY PIPES AND SQUARE FEET OF DIRECT RADIATION THEY WILL SUPPLY WITH 3 POUNDS STEAM PRESSURE.*

Diameter of Pipe, in Inches	Distance of Radiator from Boiler, in Feet.						
	9	64	100	225	324	400	484
	Sq. ft.	Sq. ft.	Sq. ft.	Sq. ft.	Sq. ft.	Sq. ft.	Sq. ft.
¾	240	90	72	48	40	36	32
1	494	185	148	98	82	74	68
1¼	863	324	259	172	144	129	118
1½	1,361	510	408	272	226	204	185
2	2,796	1,049	839	559	466	419	381
2½	4,884	1,831	1,465	977	814	732	666
3	7,700	2,887	2,310	1,540	1,283	1,155	1,050
3½	11,323	4,246	3,097	2.264	1,887	1,698	1,544
4	15,819	5,932	4,745	3,164	2,636	2,372	2,157
4½	21,226	7,959	6,368	4,245	3,537	3,184	2,894
5	27,997	10,361	8,289	5,599	4,666	4,144	3,768
6	44,230	16,586	13,269	8,846	7,372	6,634	6,031
7	64,013	24,005	19,204	12,802	10,668	9,602	8,729
8	89,615	33,605	26,884	17,923	14,936	13,442	12,220
9	120,275	45,103	36,082	24,055	20,046	18,041	16,401
10	156,277	58,604	46,883	31,255	26,046	23,441	21,310

* F. Schumann, C.E.

USEFUL MEMORANDA: HOT-WATER HEATING.

MEASUREMENT OF FLOW AND RETURN PIPES.

For the purpose of ascertaining the amount of heating surface in flow, return pipes, and risers, the following table is used.

		Surface of Pipe Covering ¾ Inch Hair-felt and Canvas.		Table of Quantity of Water contained in 100 Lineal Feet of Pipe of Different Diameters.	
Size of Pipe.	Square Feet in One Lineal Foot.	Size of Pipe.	Multiply Length by	Diameter of Pipe.	Contents in 100 Feet in Length.
Inches.		Inches.		Inches.	Gallons.
¾	.27	1	.79	1	4.50
1	.34	1¼	.96	1¼	7.75
1¼	.43	1½	1.04	1½	10.59
1½	.50	2	1.09	2	17.43
2	.62	2½	1.20	2½	24.80
2½	.75	3	1.37	3	38.38
3	.92	3½	1.49	3½	51.36
3½	1.05	4	1.64	4	66.13
4	1.17				

To obtain the surface, multiply length of pipe by figures given in the table, always pointing off two places.

Example: 500 lineal feet 1-inch pipe multiplied by .34 equals 170 sq. ft.

EQUALIZATION OF PIPE AREAS.

(R. C. Carpenter.)

Diam. of Pipes, Inches.	Number of Small Pipes Required to Make Area Equivalent to One Larger Pipe with Allowance for Friction.											
	½ In.	¾ In.	1 In.	1¼ In.	1½ In.	2 In.	2½ In.	3 In.	3½ In.	4 In.	4½ In.	5 In.
½	1	2.0	3.7	7.6	11.3	19	37	55	80	108	146	188
¾	....	1	1.8	3.7	5.4	9.2	16.7	25.5	39	53	70	90
1	....	....	1	2.0	3.1	5.1	9.3	14.7	27	30	39	53
1¼	....	....	....	1	1.5	2.6	4.5	7.3	10.6	14.7	19.5	25
1½	....	....	....	....	1	1.7	3.1	4.7	7.1	9.8	13.4	16.8
2	....	....	....	....	....	1	1.83	2.9	4.1	5.8	7.8	9.9
2½	....	....	....	....	....	....	1	1.7	2.5	3.5	4.7	5.9
3	....	....	....	....	....	....	.....	1	1.5	2.4	2.7	3.5
3½	....	....	....	....	....	....	.....	....	1	1.4	1.8	2.5
4	....	....	....	....	....	....	.....	....	....	1	1.3	1.7
4½	....	....	....	....	....	....	.....	....	....	.....	1	1.25
5	....	....	....	....	....	....	.....	....	....	.....	.....	1

EQUALIZATION OF PIPE AREAS.

(Babcock and Wilcox.)

Diam. of Pipes, Ins.*	Number of Smaller Pipes Equivalent to One Larger Pipe.											
	¾ In.	1 In.	1½ In.	2 In.	3 In.	4 In.	5 In.	6 In.	7 In.	8 In.	9 In.	10 In.
½	2.27	4.88	15.8	31.7	96.9	205	377	620	918			
¾	1	2.05	6.9	14	42.5	90.4	166	273	405	569	779	
1	....	1	3.5	6.8	20.9	44.1	81.1	133	198	278	380	536
1½	....	....	1	1.3	6.1	13	23.8	39.2	58.1	81.7	112	157
2	....	....	....	1	3.1	6.5	11.9	19.6	29.0	40.8	55.8	78.5
2½	....	....	....	....	1.8	3.87	7.1	11.7	17.4	24.4	33.4	47.0
3	....	....	....	....	1	2.12	3.9	6.4	9.5	13.3	20.9	23.7
4	....	....	....	....	....	1	1.8	3	4.5	6.3	8.6	12.1
5	....	....	....	....	....	....	1	1.6	2.4	3.4	4.7	6.6
6	....	....	....	....	....	....	.....	1	1.5	2.1	2.8	4.0
7	....	....	....	....	....	....	.....	.....	1	1.4	1.9	2.7
8	....	....	....	....	....	....	.....	.....	.....	1	1.3	1.9

* Nominal diameters standard steam- and gas-pipes.

EXAMPLE.—To find number of 2-inch pipes to deliver as much fluid as one 5-inch pipe: In column headed 5, and opposite 2, read 9.9 upper table and 11.9 lower table, the equivalent number of 2-inch pipes.

LAP-WELDED CHARCOAL-IRON BOILER TUBES.

Standard Dimensions.

(Table of Morris, Trasker & Co., Inc.)

External Diameter.	Internal Diameter.	Standard Thickness.	Birmingham Wire Gauge.	Internal Circumference.	External Circumference.	Internal Area.		External Area.		Length of Tube per Square Foot, Inside Surface.*	Length of Tube per Square Foot, Outside Surface.*	Length of Tube per Square Foot, Mean Surface.*	Weight per Lineal Foot.
Ins.	Ins.	Ins.	No.	Ins.	Ins.	Sq. Ins.	Sq. Ft.	Sq. Ins.	Sq. Ft.	Feet.	Feet.	Feet.	Lbs.
1	.810	.095	13	2.545	3.142	.515	.0036	.785	.0055	4.479	3.820	4.149	.90
1¼	1.060	.095	13	3.330	3.927	.882	.0061	1.227	.0085	3.604	3.056	3.330	1.15
1½	1.310	.095	13	4.115	4.712	1.348	.0094	1.767	.0123	2.916	2.547	2.732	1.40
1¾	1.560	.095	13	4.901	5.498	1.911	.0133	2.405	.0167	2.448	2.183	2.316	1.65
2	1.810	.095	13	5.686	6.283	2.573	.0179	3.142	.0218	2.110	1.910	2.010	1.91
2¼	2.060	.095	13	6.472	7.069	3.333	.0231	3.976	.0276	1.854	1.698	1.776	2.16
2½	2.282	.109	12	7.169	7.854	4.090	.0284	4.909	.0341	1.674	1.528	1.601	2.75
2¾	2.532	.109	12	7.955	8.639	5.035	.0350	5.940	.0412	1.508	1.389	1.449	3.04
3	2.782	.109	12	8.740	9.425	6.079	.0422	7.069	.0491	1.373	1.273	1.322	3.33
3¼	3.010	.120	11	9.456	10.210	7.116	.0494	8.296	.0576	1.269	1.175	1.222	3.96
3½	3.260	.120	11	10.242	10.996	8.347	.0580	9.621	.0668	1.172	1.091	1.132	4.28
3¾	3.510	.120	11	11.027	11.781	9.676	.0672	11.045	.0767	1.088	1.019	1.054	4.60
4	3.732	.134	10	11.724	12.566	10.939	.0760	12.566	.0873	1.024	0.955	0.990	5.47
4½	4.232	.134	10	13.295	14.137	14.066	.0977	15.904	.1104	0.903	.849	.876	6.17
5	4.704	.148	9	14.778	15.708	17.379	.1207	19.635	.1364	.812	.764	.788	7.58
6	5.670	.165	8	17.813	18.850	25.250	.1750	28.274	.1963	.674	.637	.656	10.16
7	6.670	.165	8	20.954	21.991	34.942	.2427	38.485	.2673	.573	.546	.560	11.90
8	7.670	.165	8	24.096	25.133	46.204	.3209	50.266	.3491	.498	.477	.488	13.65
9	8.640	.180	7	27.143	28.274	58.630	.4072	63.617	.4418	.442	.424	.433	16.76
10	9.594	.203	6	30.141	31.416	72.292	.5020	78.540	.5454	.398	.382	.390	21.00

These tubes are also made in sizes varying by 1 inch from 10 to 21 inches.

* In estimating the effective steam-heating or boiler surface of tubes, the surface in contact with air, or gases of combustion (whether internal or external to the tubes), is to be taken.

For heating liquids by steam, superheating steam, or transferring heat from one liquid or one gas to another the mean surface of the tubes is to be taken.

WROUGHT-IRON WELDED STEAM, GAS, AND WATER PIPE—STANDARD WEIGHT.

Table of Standard Dimensions, as Manufactured by National Tube Co. and Crane Company.

	Diameter.			Thickness.	Circumference.		Transverse Areas.			Length of Pipe per Square Foot of		Length of Pipe Containing One Cubic Foot.	Nominal Weight per Foot.	Number of Threads per Inch of Screw.
	Nominal Internal.	Actual External.	Actual Internal.		External.	Internal.	External.	Internal.	Metal.	External Surface.	Internal Surface.			
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Sq. Ins.	Sq. Ins.	Sq. Ins.	Feet.	Feet.	Feet.	Pounds.	
Butt-welded.	⅛	.405	.27	.068	1.272	.848	.129	.0573	.0717	9.44	14.15	2513	.241	27
	¼	.54	.364	.088	1.696	1.144	.229	.1041	.1249	7.075	10.49	1383.3	.42	18
	⅜	.675	.494	.091	2.121	1.552	.358	.1917	.1663	5.657	7.73	751.2	.559	18
	½	.84	.623	.109	2.639	1.957	.554	.3048	.2492	4.547	6.13	472.4	.837	14
	¾	1.05	.824	.113	3.299	2.589	.866	.5333	.3327	3.637	4.635	270	1.115	14
	1	1.315	1.048	.134	4.131	3.292	1.358	.8626	.4954	2.904	3.645	166.9	1.668	11½
	1¼	1.66	1.38	.14	5.215	4.335	2.164	1.496	.668	2.301	2.768	96.25	2.244	11½
Lap-welded.	1½	1.9	1.611	.145	5.969	5.061	2.835	2.038	.797	2.01	2.371	70.66	2.678	11½
	2	2.375	2.067	.154	7.461	6.494	4.43	3.356	1.074	1.608	1.848	42.91	3.609	11½
	2½	2.875	2.468	.204	9.032	7.753	6.492	4.784	1.708	1.328	1.547	30.1	5.739	8
	3	3.5	3.067	.217	10.996	9.636	9.621	7.388	2.243	1.091	1.245	19.5	7.536	8
	3½	4.0	3.548	.226	12.566	11.146	12.566	9.887	2.679	.955	1.077	14.57	9.001	8
	4	4.5	4.026	.237	14.137	12.648	15.904	12.73	3.174	.849	.949	11.31	10.665	8
	4½	5.0	4.508	.246	15.708	14.162	19.635	15.961	3.674	.764	.848	9.02	12.34	8
	5	5.563	5.045	.259	17.477	15.849	24.306	19.99	4.316	.687	.757	7.2	14.502	8
	6	6.625	6.065	.28	20.813	19.054	34.472	28.888	5.584	.577	.63	4.98	18.762	8
	7	7.625	7.023	.301	23.955	22.063	45.664	38.738	6.926	.501	.544	3.72	23.271	8
	8	8.625	7.982	.322	27.096	25.076	58.426	50.04	8.386	.443	.478	2.88	28.177	8
	9	9.625	8.937	.344	30.238	28.076	72.76	62.73	10.03	.397	.427	2.29	33.701	8
	10	10.75	10.019	.366	33.772	31.477	90.763	78.839	11.924	.355	.382	1.82	40.065	8
	11	11.75	11.000	.375	36.91	34.55	108.4	95.03	13.37	.325	.347	1.51	45.0	8
	12	12.75	12.000	.375	40.05	37.69	127.6	113.0	14.6	.299	.318	1.27	49.0	8

WROUGHT-IRON WELDED STEAM, GAS, AND WATER PIPE—EXTRA STRONG

Table of Standard Dimensions, as Manufactured by National Tube Co. and Crane Company.

	Diameter.			Thickness.	Circumference.		Transverse Areas.			Length of Pipe per Square Foot of		Nominal Weight per Foot.	Number of Threads per Inch of Screw.
	Nominal Internal.	Actual External.	Actual Internal.		External.	Internal.	External.	Internal.	Metal.	External Surface.	Internal Surface.		
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Sq. Ins.	Sq. Ins.	Sq. Ins.	Feet.	Feet.	Pounds.	
Butt-welded.	1/8	.405	.205	.100	1.272	.644	.129	.033	.096	9.44	18.63	.29	27
	1/4	.540	.294	.123	1.696	.924	.229	.068	.161	7.07	12.99	.54	18
	3/8	.675	.421	.127	2.121	1.323	.358	.139	.219	5.66	9.07	.74	18
	1/2	.840	.542	.149	2.639	1.703	.554	.231	.323	4.55	7.05	1.09	14
	3/4	1.050	.736	.157	3.299	2.312	.866	.425	.441	3.64	5.11	1.39	14
	1	1.315	.951	.182	4.131	2.988	1.358	.710	.648	2.90	4.02	2.17	11½
	1¼	1.660	1.272	.194	5.215	3.996	2.164	1.271	.893	2.30	3.00	3.00	11½
Lap-welded.	1½	1.900	1.494	.203	5.969	4.694	2.835	1.753	1.082	2.01	2.56	3.63	11½
	2	2.375	1.933	.221	7.461	6.073	4.430	2.935	1.495	1.61	1.97	5.02	11½
	2½	2.875	2.315	.280	9.032	7.273	6.492	4.209	2.283	1.33	1.65	7.67	8
	3	3.500	2.892	.304	10.996	9.086	9.621	6.569	3.052	1.09	1.33	10.25	8
	3½	4.000	3.358	.321	12.566	10.549	12.566	8.856	3.710	.955	1.14	12.47	8
	4	4.500	3.818	.341	14.137	11.995	15.904	11.449	4.455	.849	1.00	14.97	8
	4½	5.000	4.280	.360	15.708	13.446	19.635	14.387	5.248	.764	.893	18.22	8
	5	5.563	4.813	.375	17.477	15.120	24.306	18.193	6.113	.687	.793	20.54	8
	6	6.625	5.751	.437	20.813	18.067	34.472	25.976	8.496	.577	.664	28.58	8
	7	7.625	6.625	.500	23.955	20.813	45.664	34.472	11.192	.501	.598	37.67	8
	8	8.625	7.625	.500	27.096	23.955	58.426	45.664	12.762	.443	.502	43.00	8
	9	9.625	8.625	.500	30.238	27.096	72.760	58.426	14.334	.397	.443	48.25	8
	10	10.750	9.750	.500	33.772	30.631	90.763	74.662	16.101	.355	.399	54.25	8
	12	12.750	11.750	.500	40.055	36.914	127.68	108.43	19.25	.299	.325	65.00	8

DIMENSIONS OF STANDARD DOUBLE EXTRA STRONG PIPE.

Nominal.	Actual External Diameter.	Actual Internal Diameter.	Thickness.	Metal Area.	Nominal Weight per Foot, Pounds.
½	.840	.244	.298	.507	1.7
¾	1.050	.422	.314	.726	2.44
1	1.315	.587	.364	1.087	3.65
1¼	1.660	.885	.388	1.549	5.2
1½	1.900	1.088	.406	1.905	6.4
2	2.375	1.491	.442	2.686	9.02
2½	2.875	1.755	.560	4.073	13.68
3	3.500	2.284	.608	5.524	18.56
3½	4.000	2.716	.642	6.772	22.75
4	4.500	3.136	.682	8.180	27.48
4½	5.000	3.564	.718	9.659	32.53
5	5.563	4.063	.750	11.341	38.12
6	6.625	4.875	.875	15.807	53.11
7	7.625	5.875	.875	18.555	62.35
8	8.625	6.875	.875	21.304	71.62

LAP-WELDED CASING.

8¼	8.625	8.265	.180	4.775	16.07
8¼	8.625	8.167	.229	6.040	20.10
8¼	8.625	8.082	.271	7.125	24.38
8⅝	9	8.640	.180	4.987	17.60
9⅝	10	9.577	.211	6.504	21.90
10⅝	11	10.594	.203	6.886	26.72
11⅝	12	11.594	.203	7.526	30.35
12½	13	12.457	.271	10.852	33.78
13½	14	13.432	.284	12.24	42.02
14½	15	14.416	.292	13.49	47.66
15½	16	15.416	.292	14.41	51.47

Smoke Prevention.

Prof. O. H. Landreth, in a report to the State Board of Health of Tennessee (published in *Engineering News*, June 8, 1893, and quoted by Kent, p. 712) classifies the great number of smoke-prevention devices which had been invented up to that date as follows:

(*a*) **Mechanical Stokers.**—They effect a material saving in the labor of firing and are efficient smoke preventers when not pushed above their capacity and when the coal does not cake

badly. They are rarely susceptible to the sudden changes in the rate of firing frequently demanded in service.

(*b*) **Air-flues** in side walls, bridge-wall, and grate-bars through which air when passing is heated. The results are always beneficial, but the flues are difficult to keep clean and in order.

(*c*) **Coking Arches,** or spaces in front of the furnace arched over in which the fresh coal is coked, both to prevent cooling of the distilled gases and to force them to pass through the hottest part of the furnace just beyond the arch The results are good for normal conditions, but ineffective when the fires are forced. The arches also are burned out and injured by working the fire.

(*d*) **Dead-plates,** or a portion of the grate next the furnace doors reserved for warming and coking the coal before it is spread over the grate. These give good results when the furnace is not forced above its normal capacity. This embodies the method of "coke-firing" mentioned before.

(*e*) **Down-draught Furnaces,** or furnaces in which the air is supplied to the coal above the grate and the products of combustion are taken away from beneath the grate, thus causing a downward draught through the coal, carrying the distilled gases down to the highly heated incandescent coal at the bottom of the layer of coal on the grate. This is the most perfect manner of producing combustion and is absolutely smokeless.

(*f*) **Steam-jets** to draw air in or inject air into the furnace above the grate, and also to mix the air and the combustible gases together. A very efficient smoke preventer, but one liable to be wasteful of fuel by inducing too rapid a draught.

(*g*) **Baffle-plates** placed in the furnace above the fire to aid in mixing the combustible gases with the air.

(*h*) **Double Furnaces,** of which there are two different styles, neither of which have proved practical.

Among the devices which seem to have proven both practical and effective are those of the Smoke Prevention Company of America and of the American Stoker Company.

Ventilation.

Ventilation as applied to a room or building consists in supplying pure air to dilute and drive out that which has become vitiated.

Perfect ventilation consists in supplying an adequate amount of fresh air warmed or cooled to a comfortable temperature in such a manner that the circulation shall be constant and thorough in all parts of the room or building and at the same time without the creation of draughts.

Ventilation may be broadly classified as *Systematic* and *Non-systematic*.

Non-systematic Ventilation may be considered as including all ventilation produced without systematic provision for the admission and escape of the fresh air and power for moving the air.

All rooms in a building of ordinary construction receive some ventilation whenever the temperature of the room is above or below that of the surrounding air.

Pettenkofer found that by diffusion through the walls the air of a room in his house containing 2,650 cu. ft. was changed once every hour when the difference of exterior temperatures was 34°. With the same difference of temperature, but with the addition of a good fire in a stove, the change rose to 3,320 cu. ft. per hour. With all the crevices and openings about doors and windows pasted up air-tight the change amounted to 1,600 cu. ft. per hour.*

Prof. Carpenter says: "Even in the case of direct heating, where no air is purposely supplied for ventilation, there will be a change of air by diffusion of the air in the room which the writer has found practically met by an allowance equal to one to three changes in the cubic contents per hour."

Whenever air is introduced into a room as by ordinary indirect or hot-air heating, an equal amount of air must be driven from the room, or if air is drawn from a room, as by the draught in a fireplace, an equal amount of air must enter the room.

Heating by hot-air furnaces and by indirect steam or hot-water radiation will generally provide sufficient ventilation for private residences, especially if the principal rooms are provided with fireplaces or ventilation flues.

For Systematic Ventilation provision must be made for the admission and expulsion of the air through flues or definite openings and for *power* for moving the air.

The power for moving air for ventilating purposes is obtained in two ways: (1) by expansion due to heating, and (2) by a fan operated by an electric motor or by a steam- or gas-engine.

* Heating and Ventilating Buildings, p. 35.

Systematic ventilation also presupposes an attempt to admit a definite amount of air, and the first step in any system of ventilation would naturally be to decide upon the amount of air required.

Amount of Air Required for Ventilation.—Authorities differ greatly on this point, except that for school buildings it is generally agreed that 30 cu. ft. of air per minute for each occupant or 1,800 cu. ft. per hour should be the standard.*

For churches the same amount will give very fair ventilation, but for theatres and auditoriums which are usually more closely packed and occupied for a longer period, the air-supply should be from 2,000 to 2,500 cu. ft. per sitting per hour.

Hospitals require the greatest air dilution, and for such buildings an air-supply of from 4,000 to 6,000 cu. ft. per hour for each bed should be provided, depending upon the character of the cases treated, contagious diseases naturally requiring the greater amount.

The quantity of air required is sometimes measured by the number of times the air in a room will need to be changed, but to determine this accurately it is necessary to fall back to the supply per person.

Thus, if a schoolroom $27' \times 32'$ and $13'$ high contains fifty pupils and a teacher and 1,800 cu. ft. per hour is required per person, the total air-supply required per hour will be $1,800 \times 51$, or 91,800 cu. ft. As the capacity of the room is 11,232 cu. ft., the air in the room must be changed 8.26 times per hour to supply 30 cu. ft. of air per minute to each person.

It is seldom that the air in a room is changed oftener than four times per hour by natural ventilation.

Velocity of Entering Air.—The velocity of the air through the inlet registers or grilles should not exceed 4 to 6 ft. per second, the general allowance being 5 ft. per second when the inlet is 7 ft. or more from the floor.

Estimating Quantity of Air.—The quantity of air passing through a flue or opening is measured by multiplying the sectional area of the flue, or the net area of opening, in square feet by the velocity.

Thus with a velocity of 5 ft. per second the quantity of air passing through an unobstructed opening 1 ft. square will

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* This amount is required by law in Massachusetts.

equal 5 cu. ft. per second, 300 cu. ft. per minute, or 1,800 cu. ft, per hour.

Velocities of air are measured by an instrument called an anemometer.

Location of Inlet and Outlet.—Mr. W. R. Briggs, of Bridgeport, Conn., some years ago demonstrated quite conclusively that in a rectangular room of moderate size the best results are obtained when the inlet is in an inner wall near the ceiling and the outlet is nearly under the inlet and close to the floor.

This is now the general practice in well-designed schoolhouses, and for churches and hospitals when warmed by indirect radiation.

For cubical rooms not exceeding 50 ft. square the author considers that one inlet is better than several.

In the ventilation of theatres the air is sometimes admitted through the ceiling, but more often through the risers of the floor or through specially designed seat ends.

Size of Flues.—The size of the flues both for inlet and outlet is determined by the quantity of air to be moved and the velocity, or

$$\left.\begin{array}{c}\text{Sectional area of}\\ \text{flue in sq. ft.}\end{array}\right\} = \left\{\frac{\text{Quantity of air in cubic feet per minute}}{\text{Velocity in feet per minute.}}\right.$$

The actual velocity will depend upon the motive power, the length, size, shape, and surface of the flue, the number of turns or offsets, and whether the flue is vertical or horizontal so that after the theoretical size of the flue has been determined, the actual size will oftentimes need to be increased by an amount which must be determined largely by the judgment of the designer. For this reason considerable practical experience with forced hot-air heating and ventilating is required to lay out the system of flues to the best advantage, and the architect when designing such a plant will do well to secure the assistance of an expert.

With fan systems of ventilation the inlet openings should be of such size that the required amount of air may be introduced with a velocity not exceeding 500 ft. per minute when the inlet is 5 ft. from the floor or 288 ft. per minute when the inlet is in the floor or in the walls near the floor.

In figuring the size of ordinary registers the required area should be increased about 50 per cent. to allow for the grilles

or pattern. With light grilles of $\frac{1}{16}'' \times \frac{5}{16}''$ iron an allowance of 10 per cent. will ordinarily be sufficient.

The velocity in *vertical flues* supplying the inlets should not exceed that through the opening by more than 50 per cent., which gives a velocity in the vertical flues of from 500 to 800 ft. per minute. The rate of flow through the connections to the base of the flues should in turn be higher than that through the flues, while the velocity in the main horizontal distributing ducts should be still higher. "In fact, in schools and churches the plan should be to gradually reduce velocities from the point of leaving the fan to the point of discharge to the rooms. Careful investigation has shown that, everything considered, the velocity in the main horizontal ducts from the fan should not fall below 1,500 ft. per minute and preferably 2,000 ft. per minute." *

The size of vent or eduction flues when air is forced into the rooms by a fan should be two thirds to three fourths the sectional area of the induction flues.

Velocity of Air in Vertical Flues Due to Expansion by Heat.—The velocity of air in heated flues is dependent upon the excess of temperature of the air in the flue above that of the room or space into which the flue empties, the height of the flue, the loss by friction, and the pressure which must be resisted by the entering air. Thus in a room heated by indirect radiation or a warm-air furnace if no provision is made for ventilation the heated air must force its way into the room, pushing out an equal volume of air around doors or windows, while if there is a good ventilating flue the movement of air into the warm-air flue is assisted. The table on opposite page, quoted by various writers, shows the velocities † of air that may be expected in vent flues under the conditions noted.

To obtain the cubic feet of air discharged per *hour* per square foot of cross-section of the flue, multiply the figures in the table by 60.

While this table does not strictly apply to flues conveying warm air into a room it is sufficiently accurate for practical purposes.

Prof. Carpenter says that in residence heating the velocity in flues is likely to be as follows, in feet per minute: First story,

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* Ventilation and Heating. B. F. Sturtevant Co.

† The velocity in a flue 1 foot square being the same as the quantity of air discharged.

TABLE SHOWING THE QUANTITY OF AIR, IN CUBIC FEET, DISCHARGED PER MINUTE THROUGH A FLUE OF WHICH THE CROSS-SECTIONAL AREA IS ONE SQUARE FOOT.

(EXTERNAL TEMPERATURE OF THE AIR, 32° FAHR.; ALLOWANCE FOR FRICTION, 50 PER CENT.)

Height of Flue, in Feet.	Excess of Temperature of Air in Flue above that of External Air.							
	10°	15°	20°	25°	30°	50°	100°	150°
1	34	42	48	54	59	76	108	133
5	76	94	109	121	134	167	242	298
10	108	133	153	171	188	242	342	419
15	133	162	188	210	230	297	419	514
20	153	188	217	242	265	342	484	593
25	171	210	242	271	297	383	541	663
30	188	230	265	297	325	419	593	726
35	203	248	286	320	351	453	640	784
40	217	265	306	342	375	484	684	838
45	230	282	325	363	398	514	724	889
50	242	297	342	383	419	541	765	937
60	264	325	373	420	461	594	835	1006
70	286	351	405	465	497	643	900	1115
80	306	375	453	485	530	688	965	1185
90	324	398	460	516	564	727	1027	1225
100	342	420	485	534	594	768	1080	1325
125	383	468	542	604	662	855	1210	1480
150	420	515	596	665	730	942	1330	1630

150 to 240; second story, 300; third story, 360; fourth story, 420. Also that in usual conditions of residence heating the temperature of the air in the supply flues averages about 30° above the temperature of the air in the room.

Shape and Material of Air-ducts.—The smoother the surface of a flue the less will be the friction of the air against it and the greater the velocity. Hot- or warm-air flues should always be made of metal, preferably galvanized iron for flues exceeding 12 ins. in diameter. Brick flues should be lined with tin or galvanized iron when they convey warm air, not only to reduce the friction but also to lessen the cooling of the air. When brick flues are used for ventilation lining is not so necessary, although it will materially increase the draught.

Regarding the shape of the flue or duct, round pipes are the best, square pipes next best, and rectangular pipes should always be made as nearly square as possible.

With indirect or natural systems of ventilation each inlet register should be supplied by a separate pipe from the heater, and but one pipe should be taken from a steam or hot-water stack.

With forced systems of warming and ventilation all of the air from the heater often enters one large main, from which distributing pipes are taken off to supply the risers to the registers. With this system no branches should leave the mains at right angles, but should branch off at an angle of 45° with easy radius curves in all cases. No 90° elbow should be made with less than seven pieces or less inside radius than the diameter of the pipe. No 45° elbow should be made of less than four pieces. Each and every branch air-duct to flues should have a damper near base of flue, and at every "Y" in the system there should be placed a regulating damper. All of these dampers and fenders should be adjustable. Upon completion of the system, these dampers should be adjusted by trial so that each register will receive its proportionate supply of air and then "set."

All warm-air pipes should be covered with one or more thicknesses of asbestos paper to reduce loss of heat.

Natural Systems of Heating and Ventilating.—All systems in which the air moves upwards, due to the expansion produced by its own heat, are commonly classified as natural systems.

With such systems the ventilation is sometimes produced by *aspirating shafts* or large flues containing a heater of some sort at its base to increase the temperature of the air in the flue and thus increase the velocity. Except where they can be heated without additional cost, aspirating shafts are not as economical, as a rule, as fans.

Buildings containing but one large room can generally be fairly well ventilated by using a heavy galvanized-iron smoke-flue for the furnace or boiler and locating the flue in the centre of a large brick chimney, utilizing the space around the flue for ventilation. The heat which escapes from the flue will cause a good draught and without additional cost.

A draught may also be produced in a vent flue by means of coils of steam-pipes placed in the flue just above the air-inlet, or a gas heater may be employed for heating the flue.

The draught produced by aspiration is not usually sufficient to draw air any distance through horizontal ducts.

Natural systems of ventilation are only effective when used in connection with warming and afford no ventilation in warm weather.

One of the most effective ways of warming and ventilating without a fan system is by means of indirect steam radiation, which may be supplemented, if the room is very large, by sufficient direct radiation to offset the heat lost through the walls and windows or a total direct radiating surface equal to one fourth the sum of the glass area plus one fourth the exposed wall surface. The indirect radiation surface required can be estimated by the data given on p. 1163. A good arrangement for the indirect radiation and flues in a church or schoolhouse is illustrated in "Churches and Chapels," p. 133.

The author has obtained good results in warming and ventilating schoolrooms by hot-air furnaces, using a furnace for every two rooms and vertical vent flues for each room extending straight up through the roof. There are but few furnaces made, however, that will give satisfaction for this class of work; they should be of the horizontal tubular pattern with large radiating surface in proportion to the grate area and set in brick with a large air-chamber. An excellent furnace of this type is made by Lewis & Kitchen, of Kansas City and Chicago.

Fan Systems of Ventilation.—Ventilation by means of a fan may be effected by either of two systems, (*a*) *The Plenum System*, in which the air is forced into the room to be warmed and ventilated, and (*b*) *The Exhaust System*, in which the air is exhausted from the room.

The Exhaust System.—There are many objections to the adoption of this system, and as a rule it should be avoided when the plenum method can possibly be used. With the exhaust system a partial vacuum is created within the room and all currents and leaks are inward, so that air rushes around doors and windows, forming unpleasant and sometimes dangerous currents of air. The circulation of the air in the room is also less thorough when exhausted than when forced in. The exhaust system as a rule is used principally for affording ventilation in hot weather or for removing disagreeable odors, dust, etc., for which purpose it is both economical and effective when properly installed.

An exhaust fan can also be used to advantage for ventilating churches in connection with hot-air furnaces or indirect steam radiation, as it can be used both in winter and summer and

for as short a time as may be needed. The ventilation required in a church varies greatly at different times; a church seating 500 persons cannot be sufficiently ventilated when every seat is occupied without a fan, while when there are only one or two hundred people present, a fan may not be required. By this system the fan should be placed in the top of the main ventilation shaft or in a tower or ventilating chamber under the roof, with ducts leading to the outlet registers, and should be operated by electricity.

The Plenum, or Hot-blast system, on the other hand, maintains a slight pressure in the room or rooms ventilated and the leakage is outward instead of inward. By this system the temperature of the air and point of admission are completely under control. The denser the air also up to a certain limit the better it is for comfort and good acoustics.

For heating and ventilating theatres, hospitals, and large schools and churches this is undoubtedly the best system that can be employed, and, with the possible exception of churches, is as economical of fuel and maintenance as an indirect steam-heating plant, while affording superior ventilation and greater comfort. This system has also been applied to office buildings, factories, and buildings used for various purposes. The system may be used in summer as well as in winter, and by providing a cooling chamber, the air may be cooled to any desired temperature.

As ordinarily installed a forced-blast system consists of a heater and fan with flues and ducts for conveying the air to the various apartments as explained on p. 1153, and the entire apparatus with the exception of the vertical flues is usually located in the basement.

Two systems of ducts are commonly employed, viz., the single-duct and double-duct system. A typical arrangement of the single-duct system is shown by Fig. 35. The fan is located at one side of the fresh-air chamber, so that air is drawn into it at *A* and is forced through the heater into a warm-air chamber from which one large duct with distributing branches is taken off. A by-pass is provided so that a portion of the air passes under the heater without being warmed, and by means of a damper at the mouth of the duct more or less of the cool air may be mixed with the heated air as desired.

With this system all of the air conveyed through the ducts is of the same temperature.

With the double-duct system the upper duct conveys only warm air and the under duct cool air, and the mixing damper is placed at the bottom of the riser to each outlet. By this system the temperature of the air to each room may be regulated independently of the others.

A modification of the single-duct system is commonly used in heating schools in which a large double chamber is located near the heating stack, one portion being at all times filled with warm air and the other portion with cool air. From this double chamber a single duct is led to each room, and the connection is made with the chamber in such a way that either

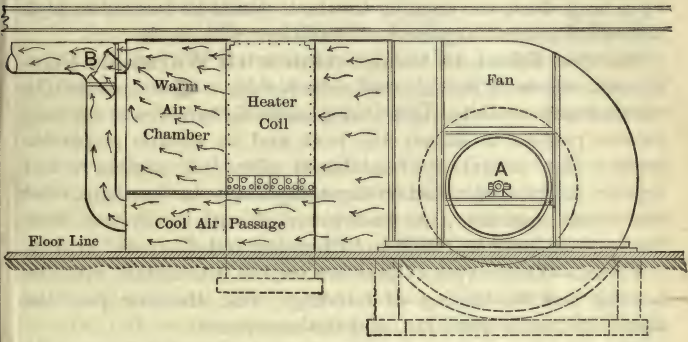

Fig. 35

all warm or all cool air, or any proportion of both, may be admitted into the duct, the mixing being controlled by a damper operated by a thermostat placed in the room with which the duct connects. This arrangement saves the cost of running two pipes, and when a thermostat regulating apparatus is used to control the dampers is the most practical system.

When there are several rooms to be warmed and a thermostatic regulating apparatus is not employed, so that the mixing dampers must be operated by hand, the double-duct system should be employed.

The system shown by Fig. 35 answers very well for warming churches and auditoriums

The various systems of piping are fully described in the catalogues of the companies named on p. 1153.

When the fan is to be run in warm weather provision should

be made so that the entire capacity of the air may pass around the heater.

By the arrangement illustrated in Fig 35 the fan is placed between the heater and the cold-air chamber and *forces* the air through the heater. The fan may, however, be placed on the other side of the heater so as to *pull* the air through it by exhaustion, at the same time forcing the heated air into the ducts. Both arrangements are used, but the former is the one more commonly employed.

With the forced-blast systems of warming and ventilating a fresh-air chamber of ample size must be provided adjacent to the fan or heater and communicating with the outside air by a large duct, the opening to which should be located as high above the ground as practical conditions will admit.

Forced Blast in Connection with Warm-air Furnaces.—Several schools and churches have been successfully warmed and ventilated by utilizing warm-air furnaces of the long tubular pattern to supply the heat and an electric motor for power. For churches of moderate size this system would appear to have some advantages, especially in economy, over the steam systems. A description of such a system with illustrations may be found in "Churches and Chapels," p. 148.

Fans.—Three types of fans are used in connection with the heating and ventilating of buildings, viz., the *disc fan*, the *blower*, or *paddle-wheel fan*, and the *cone fan*.

The disc fan receives the air at one side and delivers it at the opposite side, the principal motion of the air being parallel with the axis. This type is only used for exhausting air, and is commonly used for ventilating single rooms in warm weather. Most of the electric fans used for ventilating kitchens, restaurants, etc., are of this type.

The paddle-wheel fan is the type commonly used with the forced-blast systems of heating. The fan in steel-plate blowers is of the paddle-wheel type.*

The cone fan is a special type of the paddle-wheel fan which has been used for maintaining a plenum in a large chamber under an audience room. It is not adapted to high pressures.

Fans may be driven from a running countershaft, from an engine directly connected, or from an electric or water motor.

Disc fans are commonly driven by an electric motor, and this will be found the most convenient power for driving steel-

* The three types of fans are illustrated in "Churches and Chapels."

plate blowers in churches and theatres, as in the summer-time no heat is required. In schools, which are not used much in warm weather, and in buildings where steam is kept up all the year round, a small steam-engine will generally be most economical.

All fans make some noise, hence they should be located where they will be heard the least.

Capacity of Fans. — The catalogued capacities of all makes of fans are their capacities when running light in the open air, not being attached to any ducts or heating coils. These capacities will be reduced from 25 to 50 per cent. when so attached, depending on the length of the ducts and the method of distribution.

In figuring capacity of fans for forcing air through heating coils and ducts it is customary to call the peripheral velocity of the fan blades equal to the linear velocity of the air, and to take *one half* of the theoretical delivery as the actual efficiency.

The peripheral velocity is obtained by multiplying the revolutions per minute by the circumference of the wheel.

Thus a fan 6 ft. in diameter running 200 revolutions per minute has a peripheral velocity $= 200 \times 18.84 = 3{,}768$ ft. per minute. Deducting 50 per cent. for loss, the actual velocity of the air would be 1,884 ft. per minute. The discharge opening in a fan 6 ft. in diameter will have an area of at least 11.5 sq. ft. Multiplying this area by the working velocity we have 21,666 cu. ft. per minute as the probable actual discharge of the fan.

Mr. F. R. Still, of Detroit, who has had extensive engineering experience with forced-blast systems, says that the *maximum limit of speed* of a blower without making a serious noise is 250 revolutions per minute, and that except in rare cases the blower should run at from 180 to 200 revolutions per minute.

With a disc fan, used for ventilation only, the velocity should never exceed 900 ft. per minute.

As explained above, the actual capacity when connected with a heating and ventilating system will be reduced from 25 to 50 per cent. from the values in the table on the next page, while the horse-powers, on the other hand, are probably somewhat in excess of those actually required.

For further information on this subject the reader is referred to the catalogues of the various manufacturers of blowers and to "Heating and Ventilating Buildings."

TABLE OF CAPACITY AND POWER REQUIRED FOR STEEL-PLATE BLOWERS OF VARIOUS SIZES.

WITH FREE INLET AND OUTLET.

Size, Inches.	Diameter of Wheel, Inches.	¼-ounce Pressure.			½-ounce Pressure.		
		Revolutions.	Cubic Feet per Minute.	H.P.	Revolutions.	Cubic Feet per Minute.	H.P.
70	42	214	10,336	.3	312	14,628	1.3
80	48	188	12,584	.5	265	17,809	1.6
90	54	167	16,150	.7	236	22,856	2.0
100	60	150	20 723	.9	212	29,329	2.6
110	66	137	24,548	1.1	193	34,741	3.1
120	72	125	30,165	1.3	177	42,678	3.8
140	84	107	40,465	1.8	152	57,268	5.1
160	96	94	51,344	2.3	133	72,264	6.4

Size, Inches.	Diameter of Wheel, Inches.	¾-ounce Pressure.			1-ounce Pressure.		
		Revolutions.	Cubic Feet per Minute.	H.P.	Revolutions.	Cubic Feet per Minute.	H.P.
70	42	377	17,928	1.6	428	20,700	3.7
80	48	325	21,827	2.4	367	25 202	4.5
90	54	289	28,012	3.7	333	32,343	5.7
100	60	260	35,945	4.8	300	41,503	7.4
110	66	236	42,579	5.7	273	49,162	8.8
120	72	217	52,304	7.0	250	60,392	10.7
140	84	186	70,188	9.4	214	81,040	14.4
160	96	163	89,057	11.5	152	102,807	18.3

Chimneys.

Object.—A chimney is required for two purposes, (1) to produce the draught necessary for the proper combustion of the fuel, and (2) to furnish a means of discharging the noxious products of combustion into the atmosphere at such a height from the ground that they may not prove a nuisance to people living in the vicinity of the chimney.

A good draught is absolutely essential to the satisfactory and economical working of either a heating or power plant. It is claimed by Kent that chimneys over 150 ft. in height are not justified from the standpoint of economy, but where the gases of combustion are poisonous, as in the case of smelters, or

specially noxious, tall chimneys enhance the value of surrounding property, if in a town, far more than the cost of the chimney, and should be required by law.

Theory of Chimneys.*—To produce an effective draught in the furnace a chimney requires *size* and *height.*

Each pound of coal burned yields from 13 to 30 lbs. of gas the volume of which varies with the temperature.

The Weight of Gas carried off by a chimney in a given time depends upon three things—size of chimney, velocity of flow, and density of gas. But as the density decreases directly as the absolute temperature, while the velocity increases, with a given height, nearly as the square root of the temperature, it follows that there is a temperature at which the weight of gas delivered is a maximum. This is about 550° above the surrounding air. Temperature, however, makes so little difference that at 550° above, the quantity is *only four per cent.* greater than at 300°. Therefore height and area are the only elements necessary to consider in an ordinary chimney.

The Intensity of Draught is, however, independent of the size, and depends upon the difference in weight of the outside and inside columns of air, which varies directly with the product of the height into the difference of temperature. This is usually stated in an equivalent column of water and may vary from 0 to possibly 2 ins.

To Find the Maximum Draught for any given chimney, the heated column being 612° F. and the external air 62°: *Multiply the height above grate in feet by* **.0075** *and the product is the draught power in inches of water.*

The intensity of **draught required** varies with the kind and condition of the fuel and the thickness of the fires. Wood requires the least and fine coal or slack the most. To burn anthracite slack to advantage, a draught of $1\frac{1}{4}$ ins. of water is necessary, which can be attained by a well-proportioned chimney 175 ft. high.

A **round chimney** is better than square and a straight flue better than tapering, though it may be either larger or smaller at top without detriment.

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* Babcock & Wilcox Co.

Size of Chimneys for Power Plants.*

The effective **area of a chimney** for a given power varies inversely as the square root of the height. The actual area, in practice, should be greater, because of retardation of velocity due to friction against the walls. On the basis that this is equal to a layer of air 2 ins. thick over the whole interior surface, and that a commercial horse-power requires the consumption of an average of 5 lbs. of coal per hour, we have the following formulæ:

$$E=\frac{0.3H}{\sqrt{h}}=A-0.6\sqrt{A}; \quad (1)$$

$$H=3.33E\sqrt{h}; \quad (2)$$

$$S=12\sqrt{E}+4; \quad (3)$$

$$D=13.54\sqrt{E}+4; \quad (4)$$

$$h=\left(\frac{0.3H}{E}\right)^2. \quad (5)$$

In which H = horse-power; h = height of chimney in feet; E = effective area, and A = actual area in square feet; S = side of square chimney and D = dia. of round chimney in inches. The first table on the next page was calculated by means of these formulæ.

High Chimneys Not Necessary.†—"Chimneys above 150 ft. in height are very costly and their increased cost is rarely justified by increased efficiency. In recent practice it has become somewhat common to build two or more smaller chimneys instead of one large one. A notable example is the Spreckles sugar refinery in Philadelphia, where three separate chimneys are used for one boiler plant of 7,500 H.P. The three chimneys are said to have cost several thousand dollars less than a single chimney of their combined capacity would have cost."

Size of Chimneys for House Heaters.—Chimney-flues for heating apparatus should be ample in size and carried as nearly straight as possible from a point near the cellar floor to above the highest projection of the roof. They should be independent, having no connection with other flues or openings, and always of the same area from top to bottom. A well-

* These formulæ are those given by Kent, and are generally accepted as reliable.

† Kent.

SIZES OF CHIMNEYS WITH APPROPRIATE HORSE-POWER OF BOILERS.

Diameter in Inches.	Effective Area, Square Feet.	Actual Area, Square Feet.	Height of Chimneys.											Side of Square Flue, Inches.
			50 Ft.	60 Ft.	70 Ft.	80 Ft.	90 Ft.	100 Ft.	110 Ft.	125 Ft.	150 Ft.	175 Ft.	200 Ft.	
			Commercial H.P. of Boiler.											
18	0.97	1.77	23	25	27	...	...	...	....	....	....	....	....	16
21	1.47	2.41	35	38	41	...	...	...	....	....	....	....	....	19
24	2.08	3.14	49	54	58	62	...	...	....	....	....	....	....	22
27	2.78	3.98	65	72	78	83	...	...	....	....	....	....	....	24
30	3.58	4.91	84	92	100	107	113	...	....	....	....	....	....	27
33	4.48	5.94	...	115	125	133	141	...	....	....	....	....	....	30
36	5.47	7.07	...	141	152	163	173	182	....	....	....	....	....	32
39	6.57	8.30	...	...	183	196	208	219	....	....	....	....	....	35
42	7.76	9.62	...	...	216	231	245	258	271	....	....	....	....	38
48	10.44	12.57	...	...	...	311	330	348	365	389	....	....	....	43
54	13.51	15.90	...	...	...	...	427	449	472	503	551	....	....	48
60	16.98	19.64	...	...	...	..	536	565	593	632	692	748	....	54
66	20.83	23.76	...	...	...	...	...	694	728	776	849	918	981	59
72	25.08	28.27	...	...	...	...	...	835	876	934	1023	1105	1181	64
78	29.73	33.18	...	...	...	...	...	...	1038	1107	1212	1310	1400	70
84	34.76	38.48	...	...	...	...	...	...	1214	1294	1418	1531	1637	75
90	40.19	44.18	...	...	...	...	...	...	....	1496	1639	1770	1893	80
96	46.01	50.27	...	...	...	...	...	...	....	....	1876	2027	2167	86

jointed tile flue, preferably round, is better than a square brick flue of larger area. The chimney flue should be carried 3 or 4 ft. below the smoke-pipe entrance and provided with a clean-out door at the base, tightly fitted, to facilitate the removal of accumulated dust and soot.

The size of flues may be calculated from the following table:

Total Contents of Building, Cubic Feet of Space.	Average of Direct Radiation Steam, Square Feet.	Tile Flues, Standard Sizes, Square or Rectangular, Outside Dimensions.	Tile Flues, Standard Sizes, Round, Inside Dimensions.	Brick Flues, Inside Dimensions.
		Inches.	Inches.	Inches.
10,000 to 20,000	200 to 400	$8\frac{1}{2}\times 8\frac{1}{2}$	8	8×8
25,000 to 50,000	450 to 900	$8\frac{1}{2}\times 13$	10	8×12
60,000 to 100,000	1,000 to 1,600	13×13	12	12×12
100,000 to 150,000	1,600 to 3,000	18×18	16	16×16

Indirect radiation should be counted as 50 per cent. more than direct and corresponding areas of flue be provided for.

The amount of radiation determines the requisite size of boiler, and therefore the area of the flue.

No chimney-flue should be less than 8 ins. in depth, nor of a smaller size than the smoke-pipe from the heater.

For a kitchen range an 8×8 tile flue will generally answer, but an 8×12 flue is better.

For fireplaces the sectional area of the flue for burning wood or bituminous coal should be one tenth to one eighth that of the fireplace opening for a rectangular flue and one twelfth for a circular flue. For burning anthracite coal the above proportions may be reduced to one twelfth and one sixteenth respectively.

When practicable, chimneys should extend above the highest surrounding roof, to prevent down-draught caused by eddies. When this is impracticable a revolving chimney-top will often prevent down-draughts. They may also often be avoided by covering the top of the chimney with a stone flag and leaving openings in two parallel sides of the chimney, the sides parallel to the ridge of the adjoining roof or building being closed.

The walls of the flue should be as smooth as possible. Tile-flue lining is preferable. Brick flues should be either smoothly plastered on the inside with rich lime mortar or the joints should be filled full and struck with the point of the trowel. If the bricks are laid in cement mortar, the author recommends striking the joints instead of plastering.

The walls of attached chimneys with flues not exceeding 8″×12″ may be 4″ thick for heights of 50 ft. Flues 12″×12″ and larger should have walls 8″ thick to within 10 ft. of the top. Aside from strength or stability, thick walls are preferable to thin walls.

Stability of Chimneys.—A general rule for diameter of base of brick chimneys standing free, approved by many years of practice in England and the United States, is to make the diameter of the base, or side of a square chimney, one tenth of the height.

Construction of Brick Chimneys.—"For chimneys of 4 ft. in diameter and 100 ft. high and upwards, the best form is circular with a straight batter on the outside. A circular chimney of this size, in addition to being cheaper than any other form, is lighter, stronger, and looks much better and more shapely.

"Chimneys of any considerable height are not built up of uni-

form thickness from top to bottom nor with a uniformly varying thickness of wall, but the wall, heaviest of course at the base, is reduced by a series of steps.

"All boiler chimneys of any considerable size should consist of an outer stack of sufficient strength to give stability to the structure and an inner stack or core independent of the outer one. This core is by many engineers extended up to a height of but 50 or 60 ft. from the base of the chimney, but the better practice is to run it up the whole height of the chimney; it may be stopped off, say, a couple feet below the top and the outer shell contracted to the area of the core, but the better way is to run it up to about 8 or 12 ins. of the top and not contract the outer shell. But under no circumstances should the core at its upper end be built into or connected with the outer stack. This has been done in several instances by bricklayers, and the result has been the expansion of the inner core, which lifted the top of the outer stack squarely up and cracked the brickwork." *

Notwithstanding the above, a number of tall brick chimneys have been built without an interior wall, an instance of which is given on the next page.

Thickness of Walls.—The following is considered as a safe rule for the thickness of the outer wall of tall chimneys: For the first 25 ft. from the top, one brick (8 or 9 ins); for the second 25 ft., 1½ bricks, and so on, increasing one half brick for each 25 ft. from the top downwards. If the inside diameter exceeds 5 ft. the top length should be 1½ bricks, the next two bricks, etc.; if under 3 ft., the top may be one half brick for 10 ft.

The batter should be not less than 1 in 36 to give stability.

The inside core may be 4 ins. thick for 25 ft. from the top, then 8 or 9 ins. for 50 ft.

Two chimneys of the Edison station, Brooklyn, each 150 ft. high, have inner cores 80 ft. high and one brick thick for the full height, the first 50 ft. being of fire-brick.

Fire-brick Lining.—If a chimney has but one wall it should be lined with fire-brick for at least 30 ft., and if it has an inner core, the latter is usually built of firebrick for 30 or 50 ft. from the bottom.

The top of tall brick chimneys should be protected by a cast-iron cap.

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* From *The Locomotive*, 1884 and 1886.

Examples of Tall Brick Chimneys.—Several tall brick chimneys are described in the thirteenth edition of this book, also in Kent, p. 737.

Chimney-stack at the West Cumberland Hematite Iron Works.—Designed by Professor J. Macquorn Rankine, and considered as a model chimney.

Duty.—The duty of this chimney is to carry off the gaseous products of combustion from four blast-furnaces and from various stoves and boilers. The total amount of fuel consumed is estimated at about 10¼ tons per hour when all the furnaces are at work.

The actual temperature inside the chimney when doing about three fourths of its full duty is 490° F., and the pressure of the draught is 1⅞ ins. of water.

Figure and Dimensions.—Above ground the chimney is a frustum of a cone, with a straight batter. Underground there is a plinth or basement, octagonal outside at the ground line and square at the bottom; cylindrical inside and pierced with four circular openings for flues.

Height of chimney above the ground, 250 ft.

Depth of foundation below the ground, 17 ft.

Total height from foundation to top, 267 ft.

Inside diameter at top of cone, 13 ft.

Inside diameter 2 ft. above bottom of cone, 21 ft. 10 ins.

Inside diameter in basement, 18 ft. 10 ins.

Inside diameter of archway for flues, 7 ft. 6 ins.

Outside diameter at top of cone, 15 ft. 3 ins.

Outside diameter 2 ft. above bottom of cone, 25 ft. 7 ins.

Outside dimensions of square basement, 30 ft.×30 ft.

Size of foundation course, 31 ft. 6 ins.×31 ft. 6 ins.

Size of concrete foundations, 34 ft. 6 ins.×34 ft. 6 ins. and 3 ft. thick.

Thickness of Brickwork.—First 2 ft. above foundation stepping from four bricks to 2½ bricks; next 88 ft., 2½ bricks; next 80 ft., 2 bricks; remaining 80 ft., 1½ bricks.

The pressure on the ground below the concrete is 1.6 tons on the square foot.

Fire-brick Lining.—The thickness of brickwork given above included the fire-brick lining, which was one brick in thickness in the first 90 ft. and one half brick the remaining height, the fire-brick being bonded in with the common brick, but being

laid in fire-clay. This method of construction was considered better than that of the inner cone.

Strips of No. 15 hoop iron, tarred and sanded, were laid in the bed-joints of the cone at intervals of 4 ft. in height, with their ends turned down in the side-joints. The length of the iron was twice the circumference of the chimney.

Cap and Lightning Conductor.—On the top of the chimney is a pitch-coated, cast-iron curb 1 in. thick, coming down 3 ins. on the outside and inside. The lightning conductor is a copper-wire rope ¾ in. in diameter. It terminates in a covered drain, in which there is always a sufficient run of water.

Some of the Tallest Chimneys on Earth.*

	Height in Feet.
†Freiberg, Saxony, Germany, Halsbrucke Foundry	460
Glasgow, Port Dundas, Scotland, F. Townsend	454
Glasgow, St. Rollox, Scotland, Tenant & Co.	436½
Creusot, France, Messrs. Musprath Chemical Works	406
Halifax, Dean Clough Mill, Scotland, Messrs. Crossley's	381
Lancashire, Bolton, England, Dobson & Barlow	367
Boston, Mass., United States, Fall River Iron Co.	350
Chicago, Illinois, United States	350
East Newark, N. J., United States, Clark Thread Co.	335
Barmen, Prussia, Germany, Wessenfield & Co.	331
Edinburgh, Scotland, Gas Works	329
Huddersfield, England, Brook & Son, Fire-clay Works	315
Smethwick, England, Adams Soap Works	312
Carlisle, England, P. Dickon & Son	300
Bradford, England, Mitchell Brothers	300
Greenhithe, Kent, England, J. C. Johnson	297
Lowell, Mass., United States, Merrimack Mfg. Co.	283
Dundee, Scotland, Camperdown, Linen Works, Cox Bros.	282
Creusot, France, Schneider & Co.	280
Darwin, North Lancashire, Darwin & Mostyn Iron Co.	275
Pittsburg, Pennsylvania, United States	275
Lancashire, Eng., Barrow-in-Furness, Hematite Iron Co.	259
Bradford, England, Manningham Mills, Lester & Co.	256½
Manchester, N. H., United States, Amoskeag Mfg. Co.	255
West Cumberland, England, Hematite Iron Works	250
Lancaster, England, Story Brothers	250
Lawrence, Mass., Washington Mills	250
Cheshire, England, Connah's Quay, Chemical Co.	245

* This is part of a list of chimneys compiled by W. Barnet Le Van and published in *Machinery*, Sept., 1895. The order of the list has not been changed except for the insertion by the author of several additional chimneys.

† Built by H. R. Heinicke, of perforated radial bricks, and claimed to be the tallest chimney in the world.

	Height in Feet.
Bradford, England, Newland's Mill	240
Boston Navy Yard, Mass., United States	239
Providence, R. I., Narragansett E. L. Co.	238
Lawrence, Mass., United States, Pacific Mills	233
Harwich, Dovercourt, England, Pattrie & Sons	230
Lowell, Mass., United States, Fremont & Suffolk Co.	225
Woolwich Arsenal, England, Shell Foundry	224
New York City, N. Y., U. S., New York Steam Heating Co.	221
Northfleet, England, F. C. Gostling & Co.	220
* Elizabethport, N. J., Plymouth Cordage Co.	220
Ivorydale, Ohio, United States, Procter & Gamble	218
Lawrence, Mass., United States, The Tower Pacific Mills	215
Philadelphia, Pa., The Fidelity Insurance Co.	212
Dewsbury, England, Olroyd & Sons	210
Lanarkshire, England, Coltness Iron Works	210
Wilmington, Delaware, United States, City Water Works	204
Philadelphia, Penn., United States, Finley & Schlecter	202
Camden, N. J., U. S., Highland Mill, S. B. Still & Co.	202
Ironton, Ohio, United States, Etna Iron Works	200
Lamokin, Penn., United States, John M. Sharpless & Co.	200
Duluth, Minn., United States, Hartman Gen. Electric Co.	200
Passaic, N. J., Passaic Print Works	200
Creusot, France, Schneider & Co.	197
East Newark, N. J., United States, Clark's Thread Mill	192
Cleveland, Ohio, United States, Ohio Rolling Mill Co.	190
Nottingham, England, Stanton Iron Co.	190
Deepear, Sheffield, England, Fox & Co.	186
Philadelphia, Penn., United States, John Lang Paper Mills.	181
Bayonne, N. J., U. S., Lombard, Ayres & Co. Oil Refinery.	180

A few of the tall chimneys built by the Alphonse Custodis Chimney Construction Co.:

Location.	Height, Feet.	Diam. Feet.
Constable Hook, N. J., Oxford Copper Co.	365	10
Providence, R. I., Rhode Island Suburban R'y Co.	308	16
New York City, Manhattan R'y Co.	278	17
Philadelphia, Pa., Southern Elec. L't and Power Co.	275	18
Kansas City, Mo., Metropolitan St. R'y Co.	265	16
Kansas City, Mo., Armour Packing Co.	250	14
Boston, Mass., Edison Elec. Ill. Co.	250	16
New York City, Jacob Ruppert Ice Plant	250	10
Kansas City, Mo., Cons'l'd Elec. Light and Power Co.	243	10
Cleveland, Ohio, Cleveland City R'y Co.	240	13
Millinocket, Me., Great Northern Paper Co.	235	12
Weehawken, N. J., N. Y. Cen. and H. R. R. Co.	233	11
Edgewater, N. J. N. Y. Glucose Co.	225	12
Washington, D. C., St. Elizabeth's Insane Hospital	225	10

* Reinforced concrete.

Radial Block Chimneys.—Radial blocks for chimney construction have been used extensively in England, Germany, France, and Russia for many years, but their use in this country has been quite limited.

Some thirty years ago, Alphonse Custodis, of Germany, originated a method of building tall chimneys of perforated radial blocks, made from selected clays and burned at a very high temperature, and a company * was formed for the purpose of erecting chimneys by this method of construction. Since that time the company through its various agencies has built over 4,000 chimneys in all parts of the world.

The blocks are formed to suit the circular and radial lines of each section of the chimney, so that they can be laid with thin even joints and regular smooth surfaces. The blocks being much larger than common bricks there are only about half as many joints. These chimneys are always circular in plan above the base, and except for chemical works, metal refineries, furnaces, etc., with a single-shell construction. They are undoubtedly stronger and superior in every way to common brick chimneys.

H. R. Heinicke, of Chemnitz, Germany, builder of the 460-ft. stack at Halsbrücke and many tall chimneys in Europe and America, also employs radial blocks made especially for each chimney and very much resembling those described above. A branch office is maintained at 160 Fifth Ave., New York.

The Steinl Improved Chimney Construction Company of Birmingham, Ala., designs and erects an improved radial block chimney in which every block is moulded for the position it is to occupy and is tongued and grooved on the sides, so that the blocks interlock, thereby forming a ring which it would seem to be impossible to separate. The blocks are also perforated vertically so as to receive the cement when in place.

Chimneys of Reinforced Concrete.—Within the past ten years a number of tall chimneys have been built of reinforced concrete, and it seems more than probable that this material will largely supersede brick for this purpose in the future. A well-built steel-concrete chimney should be more durable than either brick or steel, and in every respect as good, while the cost of erection is less than for a brick chimney.

In July and August, 1902, a concrete-steel chimney 180 ft.

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* Alphonse Custodis Chimney Construction Company, 517 Bennett Building, New York.

high from bottom of footing and 165 ft. above floor of boiler-room was built by Mr. Carl Leonardt for the Pacific Electric Railway Company at Los Angeles, Cal., the Ransome system of construction being employed. The inner diameter is 11 ft. for the entire height. A detailed description of this chimney was published in the *Engineering Record* of April 11, 1903.

A chimney built in 1903 for the Laclede Fire-brick Mfg. Company at St. Louis, Mo., has an inside diameter of 5 ft. and a height of 130 ft. above foundation. The materials used in the construction are river sand and T bars.

Up to the height of 65 ft. the chimney consists of two independent shells, the outer being 6 ins. thick and the inner 4 ins., separated by a 3-inch air space.

At the height of 65 ft. both shells join, the inner shell continuing and tapering in proper intervals from 5 ins. to 4 ins., and finally 3 ins. at the top. The air space is connected directly above the grade, by means of four openings $4'' \times 6''$, with the outside air, which at the 65-foot height is allowed to enter from the air space to the chimney proper through round holes. This provision is to allow the inner shell which receives the direct heat to expand and contract while being protected by the outer shell against sudden cooling from the atmosphere.*

In 1900 a chimney was built on the Ransome system for the Pacific Coast Borax Company at Bayonne, N. J., 6 ft. diameter by 150 ft. high. It consists of one outer and one inner shell from grade to top, both shells being reinforced by means of twisted square rods, vertically and horizontally.

A large number of these chimneys have been constructed on the Ransome system, among which are two at South Bend, Ind., and one for the Plymouth Cordage Company at Elizabethport, N. J. The latter is 220 ft. high, with an interior diameter of 8 ft. 8 ins. It is built in two shells each having vertical ribs running contiguously in the air space.

Self-sustaining Steel Chimneys are largely coming into use, especially for tall chimneys of iron-works and power-houses from 150 to 300 ft. in height. "The advantages claimed are: Greater strength and safety; smaller space required; smaller cost by 30 to 50 per cent. as compared with brick chimneys; avoidance of infiltration of air and consequent checking of the draught, common in brick chimneys. They are usually

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* A more complete description of this chimney, with illustrations, may be found in *Cement and Engineering News* for February, 1904.

made cylindrical in shape, with a wide curved flare for 10 to 25 ft. at the bottom. A heavy cast-iron base plate is provided, to which the chimney is riveted, and the plate is secured to a massive foundation by holding-down bolts. No guys are used." *

The Philadelphia Engineering Works, which built a large number of steel-plate chimneys, published, in 1894, a pamphlet discussing the strength and stability of such chimneys and containing tables of dimensions for stacks of varying diameter and height. This company has been succeeded by the Niles-Bement-Pond Co., who confine themselves exclusively to the construction of electric traveling cranes. The following table is compiled from the pamphlet above mentioned:

SIZES OF FOUNDATIONS FOR SELF-SUSTAINING STEEL CHIMNEYS, HALF LINED.

Diameter, clear, ft...	3	4	5	6	7	9	11
Height in feet......	100	100	150	150	150	175	225
Least diam. of foundation..........	15′ 9″	15′ 3″	20′ 4″	21′ 10″	22′ 7″	25′ 9″	29′ 11″
Least depth of foundation...	6′ 6″	7′	9′	8′	9′	10′	13′
Height in feet......	......	125	200	200	250	275	300
Least diam. of foundation..........	......	17′ 6″	23′ 8″	25′ 0″	29′ 8″	33′ 6″	36′ 0″
Least depth of foundation..........	......	7′ 6″	10′	10′	12′	12′	14′

The details of a self-sustained steel-plate stack 5 ft. inside diameter and 120 ft. high above the base ring are published in *Engineering Record* for February 15, 1902.

Hydraulics.

Water is practically an incompressible liquid, weighing, at the average temperature of 62° F., 62.355 lbs. to the cubic foot and 8.335 lbs. to the gallon. These figures change slightly with changes in temperature and atmospheric pressure, and a slight variation for the same temperature will be found in different works.

Pressure of Water.—The pressure of still water in pounds per square inch against the sides of any pipe or vessel of any shape whatever is due alone to the *head*, or height of the surface of the water above the point considered pressed

* Kent, p. 740.

upon, and is equal to 0.433 lb. per square inch for every foot of head at 62° F. The fluid pressure per square inch is equal in all directions.

To find the total pressure of quiet water against and perpendicular to any surface, whether vertical, horizontal, or inclined at any angle, whether it be flat or curved, multiply together the area in square feet of the surface pressed, the vertical depth of its centre of gravity below the surface of the water, and the constant 62.4. The product will be the required pressure in pounds. This may be expressed by formula as follows:

$$P = 62.4\,A\,D,$$

in which P = the pressure in pounds of quiescent water on the surface considered;
A = the area pressed upon in square feet; and
D = the vertical depth in feet of centre of gravity of surface considered.

TABLE A.—PRESSURE IN POUNDS PER SQUARE INCH FOR DIFFERENT HEADS OF WATER.

Head, Feet.	0	1	2	3	4	5	6	7	8	9
0	......	0.433	0.866	1.299	1.732	2.165	2.598	3.031	3.464	3.897
10	4.330	4.763	5.196	5.629	6.062	6.495	6.928	7.361	7.794	8.227
20	8.660	9.093	9.526	9.959	10.392	10.825	11.258	11.691	12.124	12.557
30	12.990	13.423	13.856	14.289	14.722	15.155	15.588	16.021	16.454	16.887
40	17.320	17.753	18.186	18.619	19.052	19.485	19.918	20.351	20.784	21.217
50	21.650	22.083	22.516	22.949	23.382	23.815	24.248	24.681	25.114	25.547
60	25.980	26.413	26.846	27.279	27.712	28.145	28.578	29.011	29.444	29.877
70	30.310	30.743	31.176	31.609	32.042	32.475	32.908	33.341	33.774	34.207
80	34.640	35.073	35.506	35.939	36.372	36.805	37.238	37.671	38.104	38.537
90	38.970	39.403	39.836	40.269	40.702	41.135	41.568	42.001	42.436	42.867

The pressure for greater heads can be readily found by multiplication or addition, thus: the pressure for a head of 110 ft. is ten times that for 11. The pressure for 118 ft. is equal to the pressure for 110 ft. plus that for 8 ft.

Flow of Water in Pipes.

[Note.—Owing to the many practical and variable conditions which affect the flow of water in pipes, such as the smoothness of the pipe, number and character of the joints, bends and

valves in the pipe, to say nothing of the size and length of the pipe, all formulas for the velocity and discharge of water in and through pipes can only be considered as approximate. The following formulas and data are taken largely from the National Tube Company's "Book of Standards," 1902 edition. They agree fairly well with similar tables in "Kent" and "Trautwine," both of whom devote much space to this subject.]

The *quantity of water* passing through a given pipe is governed by the sectional area of the pipe or outlet and the mean *velocity*. The velocity depends primarily upon the *pressure* or *head*, and is greatly affected by *friction*, which again varies with the smoothness of the bore, the diameter and length of the pipe, and whatever obstructions there may be in the pipe.

Head is the vertical distance from the surface of the water in the reservoir to the centre of gravity of the lower end of the pipe when the discharge is into the air, or to the level surface of the lower reservoir when the discharge is under water.

When the pressure is produced by mechanical means, the head in feet of water may be readily determined by the following table:

TABLE B.* — FOR CONVERTING PRESSURE PER SQUARE INCH INTO FEET HEAD OF WATER.

Pressure.	0	1	2	3	4	5	6	7	8	9
0	....	2.309	4.619	6.928	9.238	11.547	13.857	16.166	18.476	20.785
10	23.0947	25.404	27.714	30.023	32.333	34.642	36.952	39.261	41.570	43.880
20	46.1894	48.499	50.808	53.118	55.427	57.737	60.046	62.356	64.665	66.975
30	69.2841	71.594	73.903	76.213	78.522	80.831	83.141	85.450	87.760	90.069
40	92.3788	94.688	96.998	99.307	101.62	103.93	106.24	108.55	110.85	113.16
50	115.4735	117.78	120.09	122.40	124.71	126.02	129.33	131.64	133.95	136.26
60	138.5682	140.88	143.19	145.50	147.81	150.12	152.42	154.73	157.04	159.35
70	161.6629	163.97	166.28	168.59	170.90	173.21	175.52	177.83	180.14	182.45
80	184.7576	187.07	189.38	191.69	194.00	196.31	198.61	200.92	203.23	205.54
90	207.8523	210.16	212.47	214.78	217.09	219.40	221.71	224.02	226.33	228.64

* Tables A and B are exact for water at 62° F. and atmospheric pressure = 14.7 lbs.

To find the velocity of water discharged from a pipe line longer than four times its diameter, knowing the head, length, and inside diameter, use the following formula,

$$v = m\sqrt{\frac{hd}{L + 54d}},$$

in which v = approximate mean velocity in feet per second;
m = coefficient from the table below;
d = diameter of pipe in feet;
h = total head in feet;
L = total length of line in feet.

VALUES OF COEFFICIENT m.

$\sqrt{\frac{hd}{L+54d}}$	Diameter of Pipe in Feet.							
	0.05	0.10	0.50	1	1.5	2	3	4
	m	m	m	m	m	m	m	m
0.005	29	31	33	35	37	40	44	47
0.01	34	35	37	39	42	45	49	53
0.02	39	40	42	45	49	52	56	59
0.03	41	43	47	50	54	57	60	63
0.05	44	47	52	54	56	60	64	67
0.10	47	50	54	56	58	62	66	70
0.20	48	51	55	58	60	64	67	70

The above coefficients are averages deduced from a large number of experiments. In most cases of pipes carefully laid and in fair condition, they should give results from 5 to 10 per cent. of the truth.

EXAMPLE.—Given the head, $h = 50$ ft.; the length, $L = 5{,}280$ ft, and the diameter, $d = 2$ ft.; to find the velocity and quantity of discharge.

Substituting these values in above formula, we get

$$\sqrt{\frac{d \times h}{L+54d}} = \sqrt{\frac{2\times 50}{5280+108}} = \sqrt{\frac{100}{5388}} = 0.136.$$

In column headed $\sqrt{\frac{hd}{L+54d}}$ find 0.10, which is the value nearest to 0.136, and look along this line until column headed "2" is reached, then read 62 as the value of coefficient m.

Then $v = 62 \times 0.136 = 8.432$ ft. per sec., the required velocity.

To find the discharge in cubic feet per second, multiply this velocity by area of cross-section of pipe in square feet.

Thus, $3.1416 \times (1)^2 \times 8.432 = 26.49$ cu. ft. per second.

Since there are 7.48 gal. in a cubic foot, the discharge in gallons per second $= 26.49 \times 7.48 = 198.2$.

The above formula is only an approximation, since the flow is modified by bends, joints, incrustations, etc. Wrought-iron

and steel pipes are smoother than cast-iron ones, thereby presenting less friction and less encouragement for deposits; and, being in longer lengths, the number of joints is reduced, thus lessening the undesirable effects of eddy currents.

To find the head in feet necessary to give a stated discharge in cubic feet, use the formula *

$$h=\frac{0.000704\,Q^2(L+54\,d)}{d^5},$$

in which h = total head in feet;
L = total length of line in feet;
d = diameter of pipe in feet;
Q = quantity of water in cu. ft. per second.

EXAMPLE.—Given the diameter of pipe, $d=0.5$ ft.; the length of pipe, $L=20$ ft.; and the quantity of water to be discharged, $q=3.07$ cu. ft. per second; to find the necessary head.

Substituting these values in the above formula,* we get

$$h=\frac{0.000704\times 9.4\times(20+27)}{(0.5)^5}$$

$$=\frac{0.000704\times 9.4\times 47}{0.03125}=9.95 \text{ ft., the required head.}$$

The following formula * is simpler and can be used when $54d$ in relation to L is so small as to be negligible:

$$h=\frac{0.000704\,Q^2\times L}{d^5}.$$

If the pipe instead of being straight has easy curves (say with radius not less than five diameters of the pipe) either horizontal or vertical, the discharge will not be materially diminished so long as the total heads and total actual lengths of pipe remain the same, but it is advisable to make the radius as much more than five diameters as can conveniently be done.

To find the diameter of a pipe of given length to deliver a given quantity of water under a given head use the following,

$$d=0.234\sqrt[5]{\frac{Q^2L}{h}},$$

* The small 5 in these formulas denotes the fifth power or root, as the case may be.

in which d = diameter of pipe in feet;
Q = cubic feet per second delivered;
L = length of line in feet;
h = head in feet.

EXAMPLE.—Given the head, $h=700$ ft.; the length of pipe, $L=3{,}000$ ft.; the quantity to be delivered, $Q=4$ cu. ft. per sec.; required the diameter of pipe necessary.

Substituting these values in the above formula,* we get

$$d=0.234\sqrt[5]{\frac{16\times 3{,}000}{700}}=0.234\sqrt[5]{68.57}=0.545 \text{ ft.}=6.54 \text{ ins.}$$

To find the diameter of pipe required to deliver a given quantity of water with a given head.

RULE: 1st, Reduce the head to feet per 100 ft.;
then, 2d, From Table C, find the discharge for the head thus obtained through a pipe 1 ft. in diameter;
then, 3d, Divide the required discharge by that obtained from Table C; then look for the quotient in the column of Table D headed "Ratio of Discharge" and opposite it, in columns 1 and 2, will be found the required diameter.

NOTE.—*The use of Tables C and D is not sufficiently correct for pipes less than* 700 *diameters long.*

EXAMPLE.—Given the head from a reservoir to point of delivery as 20 ft. in a distance of 1,860 ft., what is the diameter of a pipe to deliver 6 cu. ft. of water per second?

$$20 \text{ ft. head in } 1{,}860 \text{ ft.}=\frac{20}{18.60} \text{ ft. in } 100 \text{ ft., or } 1.075 \text{ ft. in } 100.$$

From Table C we find the discharge per second with a head of 1.136 is 3.989; for a head of 1.075 it would be about 3.8 cu. ft. Dividing required discharge (6) by 3.8, we have 1.58. From Table D the diameter of pipe having ratio of discharge = 1.58 is found to be about $14\frac{1}{2}$, therefore we must use a 15-inch pipe to obtain the required discharge. If the required discharge is in gallons divide by 7.5. to reduce to cubic feet. If in cubic feet per minute, divide by 60 to reduce to feet per second.

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* The small 5 in these formulas denotes the fifth power or root, as the case may be.

TABLE C.—THE VELOCITIES AND DISCHARGES THROUGH A STRAIGHT, SMOOTH PIPE ONE FOOT IN DIAMETER AND ONE MILE, OR 5,280 DIAMETERS, IN LENGTH.

Head in Feet per 100 Feet.	Head in Feet per Mile.	Velocity in Feet per Second.	Discharge in Cubic Feet per Second.	Discharge in Cubic Feet per 24 Hours.
.0568	3	1.13	.8914	76,982
.0758	4	1.31	1.028	88,862
.0947	5	1.47	1.150	99,403
.1136	6	1.61	1.264	109,209
.1325	7	1.74	1.366	118,022
.1514	8	1.86	1.455	125,740
.1703	9	1.96	1.539	132,969
.1894	10	2.08	1.633	141,145
.2273	12	2.27	1.782	153,964
.2652	14	2.45	1.924	166,233
.3030	16	2.62	2.057	177,724
.3409	18	2.78	2.183	188,611
.3788	20	2.93	2.301	198,806
.4735	25	3.28	2.572	222,156
.5682	30	3.59	2.819	243,604
.6629	35	3.88	3.047	263,260
.7576	40	4.15	3.267	282,288
.8523	45	4.40	3.451	298,209
.9470	50	4.64	3.638	314,352
1.136	60	5.08	3.989	344,649
1.326	70	5.49	4.311	372,470
1.515	80	5.85	4.602	397,613
1.704	90	6.23	4.900	423,435
1.894	100	6.56	5.144	444,312
2.083	110	6.87	5.395	466,128
2.272	120	7.18	5.639	487,209
2.462	130	7.47	5.866	506,822
2.652	140	7.76	6.094	526,521
2.841	150	8.05	6.322	546,048
3.030	160	8.30	6.534	564,576
3.219	170	8.55	6.715	580,176
3.408	180	8.80	6.903	596,418
3.596	190	9.04	7.100	613,440
3.788	200	9.28	7.276	628,704
4.261	225	9.84	7.696	664,848
4.735	250	10.4	8.168	705,728
5.208	275	10.8	8.482	732,844
5.682	300	11.3	8.914	769,824
6.629	350	12.3	9.621	831,168
7.576	400	13.1	10.28	888,624
8.532	450	13.9	10.91	943,056

TABLE C.—THE VELOCITIES AND DISCHARGES THROUGH A STRAIGHT, SMOOTH PIPE ONE FOOT IN DIAMETER AND ONE MILE, OR 5,280 DIAMETERS, IN LENGTH—(*Continued*).

Head in Feet per 100 Feet.	Head in Feet per Mile.	Velocity in Feet per Second.	Discharge in Cubic Feet per Second.	Discharge in Cubic Feet per 24 Hours.
9.47	500	14.7	11.50	994,032
10.41	550	15.4	12.09	1,044,576
11.36	600	16.1	12.64	1,092,096
12.30	650	16.7	13.11	1,132,704
13.25	700	17.4	13.66	1,180,224
14.20	750	18.0	14.13	1,220,832
15.15	800	18.6	14.55	1,257,408
16.09	850	19.1	15.00	1,296,000
17.04	900	19.6	15.39	1,329,696
17.99	950	20.3	15.94	1,377,216
18.94	1000	20.8	16.33	1,411,456
22.73	1200	22.7	17.82	1,539,648
26.52	1400	24.5	19.24	1,662,336
30.30	1600	26.2	20.57	1,777,248
34.08	1800	27.8	21.83	1,886,112
37.87	2000	29.3	23.01	1,988,064
47.35	2500	32.8	25.72	2,221,560
56.81	3000	35.9	28.19	2,436,040

TABLE D.

Diameter of Pipe in Inches.	Diameter of Pipe in Feet.	Ratio of Discharge to that through a 1-foot Pipe with the Same Head per Mile.	Diameter of Pipe in Inches.	Diameter of Pipe in Feet.	Ratio of Discharge to that through a 1-foot Pipe with the Same Head per Mile.
1	.0833	.0020	$12\frac{1}{2}$	1.042	1.106
$1\frac{1}{2}$	.1250	.0055	13	1.083	1.221
2	.1667	.0113	14	1.167	1.470
$2\frac{1}{2}$	.2083	.0198	15	1.250	1.746
3	.2500	.0310	16	1.333	2.053
$3\frac{1}{2}$	.2917	.0453	17	1.417	2.388
4	.3333	.0643	18	1.5	2.754
$4\frac{1}{2}$	.3750	.0857	19	1.583	3.153
5	.4167	.1119	20	1.667	3.585
$5\frac{1}{2}$	.4583	.1422	21	1.75	4.051
6	.5	.1767	22	1.833	4.551
$6\frac{1}{2}$	.5417	.2159	23	1.917	5.084
7	.5833	.2600	24	2	5.649
$7\frac{1}{2}$	.6250	.3090	$24\frac{5}{8}$	2.052	6.000
8	.6667	.3631	26	2.167	6.912
$8\frac{1}{2}$	.7083	.4220	28	2.333	8.319
9	.75	.4871	30	2.5	9.822
$9\frac{1}{2}$	.7917	.5575	$30\frac{1}{4}$	2.521	10
10	.8333	.6337	32	2.667	11.6
$10\frac{1}{2}$	.8750	.7157	34	2.833	13.5
11	.9167	.8044	36	3	15.5
$11\frac{1}{2}$	.9583	.8987	38	3.167	17.8
12	1	1	40	3.333	20.2

This table also shows the relative discharging capacities of long pipes. Thus, one 12-inch pipe = two 9-inch pipes, nearly six 6-inch pipes, or thirty-three 3-inch pipes.

TABLE E.—FLOW OF WATER IN HOUSE-SERVICE PIPES.

(Thomson Meter Co.)

To find discharge in gallons multiply by 7.47.

Condition of Discharge.	Pressure in Main, Lbs. per Sq. Inch.	Discharge in Cubic Feet per Minute from the Pipe. Nominal Diameters of Iron or Lead Service Pipe in Inches.								
		½	⅝	¾	1	1½	2	3	4	6
Through 35 feet of service pipe, no back pressure.	30	1.10	1.92	3.01	6.13	16.58	33.34	88.16	173.85	444.63
	40	1.27	2.22	3.48	7.08	19.14	38.50	101.80	200.75	513.42
	50	1.42	2.48	3.89	7.92	21.40	43.04	113.82	224.44	574.02
	60	1.56	2.71	4.26	8.67	23.44	47.15	124.68	245.87	628.81
	75	1.74	3.03	4.77	9.70	26.21	52.71	139.39	274.89	703.03
	100	2.01	3.50	5.50	11.20	30.27	60.87	160.96	317.41	811.79
	130	2.29	3.99	6.28	12.77	34.51	69.40	183.52	361.91	925.58
Through 100 feet of service pipe, no back pressure.	30	0.66	1.16	1.84	3.78	10.40	21.30	58.19	118.13	317.23
	40	0.77	1.34	2.12	4.36	12.01	24.59	67.19	136.41	366.30
	50	0.86	1.50	2.37	4.88	13.43	27.50	75.13	152.51	409.54
	60	0.94	1.65	2.60	5.34	14.71	30.12	82.30	167.06	448.63
	75	1.05	1.84	2.91	5.97	16.45	33.68	92.01	186.78	501.58
	100	1.22	2.13	3.36	6.90	18.99	38.89	106.24	215.68	579.18
	130	1.39	2.42	3.83	7.86	21.66	44.34	121.14	245.91	660.36
Through 100 feet of service pipe and 15 feet vertical rise.	30	0.55	0.96	1.52	3.11	8.57	17.55	47.90	97.17	260.56
	40	0.66	1.15	1.81	3.72	10.24	20.95	57.20	116.01	311.09
	50	0.75	1.31	2.06	4.24	11.67	23.87	65.18	132.20	354.49
	60	0.83	1.45	2.29	4.70	12.94	26.48	72.28	146.61	393.13
	75	0.94	1.64	2.59	5.32	14.64	29.96	81.79	165.90	444.85
	100	1.10	1.92	3.02	6.21	17.10	35.00	95.55	193.82	519.72
	130	1.26	2.20	3.48	7.14	19.66	40.23	109.82	222.75	597.31
Through 100 feet of service pipe and 30 feet vertical rise.	30	0.44	0.77	1.22	2.50	6.80	14.11	38.63	78.54	211.54
	40	0.55	0.97	1.53	3.15	8.68	17.79	48.68	98.98	266.59
	50	0.65	1.14	1.79	3.69	10.16	20.82	56.98	115.87	312.08
	60	0.73	1.28	2.02	4.15	11.45	23.47	64.22	130.59	351.73
	75	0.84	1.47	2.32	4.77	13.15	26.95	73.76	149.99	403.98
	100	1.00	1.74	2.75	5.65	15.58	31.93	87.38	177.67	478.55
	130	1.15	2.02	3.19	6.55	18.07	37.02	101.33	206.04	554.96

Table E may also be used when pressure is in feet head of water by reducing the head in feet to pounds per square inch by Table A. Thus, if we wish the discharge per minute through a ¾-inch pipe 100 ft. long with a head of 70 ft., we find from Table A that a head of 70 ft. corresponds to a pressure of 30 lbs. per square inch, and from Table E we find the discharge through a ¾-inch pipe 100 ft. long with a pressure of 30 lbs. to be 1.84 cu. ft. per minute.

TABLE F.—FRICTION OF WATER IN PIPES BASED ON ELLIS AND HOWLAND'S EXPERIMENTS.

The following table gives the friction loss in pounds pressure per square inch for *each* 100 ft. of length in different size clean iron pipes discharging given quantities of water per minute. This friction loss is greatly increased by bends or irregularities in the pipe.

To find "friction head" in feet multiply figures by 2.3.

Gallons per Minute.	Sizes of Pipes, Inside Diameter.							
	¾-Inch.	1-Inch.	1¼ inch.	1½-inch.	2-inch.	2½-inch.	3-inch.	4-inch.
5	3.3	0.84	0.31	0.12				
10	13.0	3.16	1.05	0.47	0.12			
15	23.7	6.98	2.38	0.97	0.26			
20	50.4	12.3	4.07	1.66	0.42			
25	78.8	19.0	6.40	2.62	0.64	0.21	0.10	0.27
30	......	27.5	9.15	3.75	0.91			
35	......	37.0	12.4	5.05	1.22			
40	......	48.0	16.1	6.52	1.60	......	0.20	
45	......	......	20.2	8.15	2.02			
50	......	......	24.9	10.0	2.44	0.81	0.35	0.09
75	......	......	56.1	22.4	5.32	1.80	0.74	0.23
100	......	......	......	39.0	9.46	3.20	1.31	0.33
125	......	......	......	......	14.9	4.89	1.99	0.49
150	......	......	......	......	21.2	7.00	2.85	0.69
175	......	......	......	......	28.1	9.46	3.85	0.94
200	......	......	......	......	37.5	12.47	5.02	1.22
250	......	......	......	......	......	19.66	7.76	1.89
300	......	......	......	......	......	28.06	11.2	2.66
350	......	......	......	......	......	......	15.2	3.65
400	......	......	......	......	......	......	19.5	4.73
450	......	......	......	......	......	......	25.0	6.01
500	......	......	......	......	......	......	30.8	7.43
600	......	......	......	......	......	......	......	9.54
700	......	......	......	......	......	......	......	14.32

Water-pipe is usually tested to 300 lbs. pressure per square inch before delivery, and a hammer test should be made while the pipe is under pressure.

The usual length for each section of cast-iron water-pipe is 12 ft. 4 ins. to 12 ft. 6 ins., depending upon the depth of the socket, each length making approximately 12 ft. of pipe when laid. Pipes 2 to 4 ins. diameter are sometimes made in 8 or 9 ft. lengths.

SAFE PRESSURES AND EQUIVALENT HEADS OF WATER FOR CAST-IRON PIPE OF DIFFERENT SIZES AND THICKNESSES.

(Calculated by F. H. Lewis from Fanning's Formula.)

Thickness, Inches.	Size of Pipe, Inches.											
	4		6		8		10		12		14	
	Pressure in Lbs.	Head in Feet.	Pressure in Lbs.	Head in Feet.	Pressure in Lbs.	Head in Feet.	Pressure in Lbs.	Head in Feet.	Pressure in Lbs.	Head in Feet.	Pressure in Lbs.	Head in Feet.
7/16	112	258	49	112	18	42						
1/2	224	516	124	280	74	171	44	101	24	55		
9/16	336	774	199	458	130	300	89	205	62	143	42	97
5/8	....	....	274	631	186	429	132	304	99	228	74	170
11/16	....	....	....	....	....	....	177	408	137	316	106	244
3/4	....	....	....	....	....	....	224	516	174	401	138	316
13/16	....	....	....	....	....	....	....	....	212	488	170	392
7/8	....	....	....	....	....	....	....	....	249	574	202	465
15/16	....	....	....	....	....	....	....	....	....	....	234	538
1	....	....	....	....	....	....	....	....	....	....	266	612

Thickness, Inches.	16		18		20		24		30		36	
5/8	56	129	41	95								
11/16	84	194	66	152	51	118	30	69				
3/4	112	253	91	210	74	170	49	113	24	55		
13/16	140	323	116	267	96	221	68	157	39	90		
7/8	168	387	141	325	119	274	86	198	54	124	32	74
15/16	196	452	166	382	141	325	105	242	69	159	44	101
1	224	516	191	440	164	378	124	286	84	194	57	131
1 1/8	....	....	216	497	209	481	161	371	114	263	82	189
1 1/4	....	....	....	....	256	589	199	458	144	332	107	247
1 3/8	....	....	....	....	....	....	237	546	174	401	132	304
1 1/2	....	....	....	....	....	....	....	....	204	470	157	362
1 5/8	....	....	....	....	....	....	....	....	234	538	182	419
1 3/4	....	....	....	....	....	....	....	....	....	....	207	477

WEIGHTS OF LEAD AND GASKET FOR PIPE JOINTS.

(Dennis Long & Co.)

Diameter of Pipe.	Lead.	Gasket.	Diameter of Pipe.	Lead.	Gasket.
Inches.	Lbs.	Lbs.	Inches.	Lbs.	Lbs.
2	2.5	0.125	12	15	0.250
3	3.5	0.170	14	18	0.375
4	4.5	0.170	16	22	0.500
6	6.5	0.200	18	26	0.500
8	9.0	0.200	20	33	0.625
10	13.0	0.250			

WEIGHTS, PER FOOT, OF CAST-IRON PIPES IN GENERAL USE INCLUDING SOCKET AND SPIGOT ENDS.

(Dennis Long & Co., Inc., Louisville, Ky.)

Diameter.	Thickness.	Weight per Foot.	Diameter.	Thickness.	Weight per Foot.	Diameter.	Thickness.	Weight per Foot.
Ins.	In.	Lbs.	Ins.	Ins.	Lbs.	Ins.	Ins.	Lbs.
3	3/8	12½	16	3/4	129	30	2	662
	7/16	15		7/8	152	36	7/8	334
	1/2	18		1	175		1	382
	9/16	20½	18	5/8	120		1⅛	432
	5/8	23		3/4	146		1¼	482
4	3/8	17		7/8	171		1⅜	532
	7/16	20		1	197		1½	587
	1/2	23½		1⅛	223		1⅝	632
	9/16	26¾		1¼	249		1¾	683
	5/8	30	20	11/16	148		1⅞	734
6	7/16 +	30		3/4	161		2	786
	1/2	34		7/8	190	42	1	445
	9/16	38¼		1	216		1⅛	471
	5/8	42½		1⅛	247		1¼	560
	3/4	52		1¼	276		1⅜	629
8	7/16	40		1⅜	305		1½	675
	1/2	43½		1½	334		1⅝	734
	9/16	49¾	24	3/4	191		1¾	794
	5/8	56		7/8	225		1⅞	853
	3/4	68		1	258		2	912
10	7/16	50		1⅛	293	48	1⅛	572
	1/2	54		1¼	327		1¼	637
	9/16	60		1⅜	361		1⅜	701
	5/8	68		1½	395		1½	768
	3/4	82		1⅝	430		1⅝	835
12	1/2	70		1¾	465		1¾	901
	9/16	76	30	13/16	258		1⅞	967
	5/8	82		7/8	278		2	1034
	3/4	99		1	319	60	1¼	797
	7/8	117		1⅛	360		1⅜	880
14	9/16	85		1¼	405		1½	964
	5/8	94		1⅜	448		1⅝	1049
	3/4	113		1½	489		1¾	1133
	7/8	137		1⅝	532		1⅞	1216
16	9/16	100		1¾	575		2	1300
	5/8	108		1⅞	619		2¼	1470

There is no standard weight of pipe for any given pressure.

Private Water-supply:—Pumps.

The architect is frequently required to furnish a water-supply for isolated buildings, and even in cities it is becoming quite common for manufacturing establishments and large buildings to have their own water-supply, so that some knowledge of the various methods of supplying water is requisite.

Power pumps are of so many kinds and so intricate in construction that no attempt will be made to describe them.

The Hydraulic Ram.—Where a small stream of water having a fall of 5 ft. or more flows near the premises, an hydraulic ram may be used to great advantage to furnish water for domestic purposes, or even for irrigation. The ram is operated by the pressure of the stream, and delivers water into an open tank. Water can be conveyed by a ram 3,000 ft. and elevated 200 ft., provided there is sufficient fall. The drive pipe supplying the ram should be 30 or 40 ft. long to give the necessary pressure. This is the most economical method of obtaining a water-supply, as there is no expense for maintenance except for repairs, and the cost of installation is also small.

TABLE OF ACTUAL TESTS WITH GOULD'S HYDRAULIC RAMS.*

Size of Ram.	Length of Drive Pipe. Feet.	Head or Fall of Drive Pipe. Feet.	Length of Discharge Pipe. Feet.	Height or Lift of Discharge Pipe. Feet.	Water Supplied Ram per Minute. Gallons.	Water Discharge at Point of Delivery per Minute. Gallons.	List Price.
2	70	12	100	50	2.1	.3	$9
3	70	10	200	100	2.4	.2	11
4	70	12	200	100	5.6	.5	14
5	70	13	200	100	7	.8	22
5	126	20	400	200	14	1.5	
6	70	10	100	50	12.4	2.4	40
6	125	25	400	200	18	2	
7	70	11	100	40	33	7.6	75
7	184	23	767	118	27	4.5	
8	100	12	300	100	44	4	125

* Made by N. O. Nelson Mfg. Co.

Deep Wells and Plunger pumps.—The most common method of obtaining a private water-supply is to drive a deep well until a sufficient supply of water is obtained. The depth to which a well must be driven will of course depend upon the

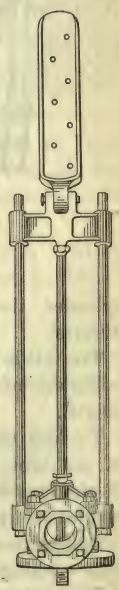
locality, and can only be determined by drillings. As the well is driven, a large wrought-iron pipe is sunk to form the casing. Casings are seldom less than 6″ inside diameter or more than 10″, 8″ being the most common size.

When the water-pocket has been reached, the water will usually rise and stand in the pipe several hundred feet above its bottom, and the amount of water that can usually be pumped from such wells, without lowering the water, is practically unlimited.

The cost of drilling deep wells, per foot of depth, *including the casing*, is approximately as follows:

Well with $6\frac{5}{8}''$ casing			$3.25 per ft.		
"	"	$7\frac{5}{8}''$ "	3.75	"	"
"	"	$8\frac{1}{4}''$ "	4.50	"	"
"	"	10″ "	5.25	"	"

For raising the water into an open tank a single-acting pump consisting of a working-head, which operates a cylinder placed in a smaller pipe lowered into the well through which the water is raised, is most commonly employed.


Working-head.

The cylinder should preferably be placed below the water-line in the well, and is usually connected with the working-head by wooden sucker-rods.

The working-head may be operated by hand, or by a crank-rod attached to a pumping-jack, windmill, or engine.

With a single-acting pump the plunger is raised and lowered once with every revolution of the driving-wheel, the principle of operation being the same as in an ordinary hand suction-pump.

The illustration on next page shows the simplest arrangement for operating a working-head by belt power (the trade term for the apparatus being "pumping-jack").

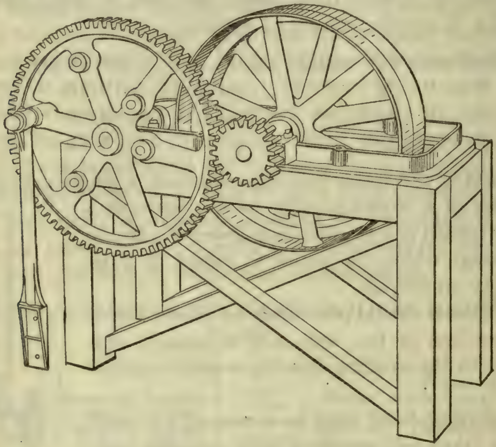
The "jack" is usually elevated some 10 or 12 ft. above the top of the well, so that a crank-rod 8 or 9 ft. long may be used to connect with the working-head which is set over the top of the well.

A more substantial arrangement is an iron frame, containing

the entire operating gear, but such a pump costs three or four times as much as a jack and working-head.

The jack shown will give variations in stroke, viz., 8, 10, 12, 16, 18, or 20 ins., by changing the connection of the crank-rod. The longer the stroke the greater will be the amount of water pumped, but it will also require more power to operate.

The amount of water pumped in a minute by any single-acting pump is determined by the diameter of the suction



Pumping-jack.

cylinder, the length of stroke, and the number of strokes per minute.

The table on opposite page gives the capacity per stroke for cylinders of different diameters, and for strokes of different length.

To find the capacity per minute, multiply the figures given in the table by the revolutions per minute. The usual speed of single-acting working-heads and pumping-jacks is 25 to 30 revolutions per minute.

Cylinders over $2\frac{3}{4}$ ins. in diameter should have a substantial iron working-head.

Hot-air Engines.—These are very extensively used for pumping water for country houses, as they are absolutely safe, require little attention, and have no valves, springs, or gauges to get out of order. They are also adapted to almost any

TABLE SHOWING CAPACITY OF SINGLE-ACTING PUMPS OF GIVEN DIAMETER AND LENGTH OF STROKE.

Diam. of Cylinder in Inches.	Length of Stroke in Inches.								
	6	8	10	12	14	16	18	20	24
			Capacity per Stroke in Gallons.						
1¼	.0319	.0425	.0531	.0637	.0743	.0848	.0955	.1062	.1274
1⅜	.0385	.0513	.0642	.077	.089	.1027	.1156	.1280	.1541
1½	.0459	.0612	.0765	.0918	.1071	.1224	.1377	.1530	.1836
1¾	.0625	.0833	.1041	.1249	.1457	.1666	.1874	.2082	.2499
2	.0816	.1088	.136	.1632	.1904	.2176	.2448	.2720	.3264
2¼	.1033	.1377	.1721	.2063	.241	.2754	.3096	.3442	.4128
2½	.1275	.17	.2125	.255	.2975	.34	.3825	.425	.51
2¾	.1543	.2057	.2571	.3085	.3598	.4114	.4626	.5142	.617
3	.1836	.2448	.306	.3672	.4284	.4896	.5508	.612	.7344
3¼	.2154	.2872	.3594	.4312	.503	.5748	.6466	.7182	.8624
3½	.2499	.3332	.4165	.4998	.5831	.6664	.7497	.833	.9996
3¾	.2868	.3824	.478	.5736	.6692	.7648	.8605	.9561	1.147
4	.3264	.4352	.544	.6528	.7616	.8704	.9792	1.088	1.3056
4¼	.3684	.4912	.6141	.7368	.8596	.9824	1.105	1.228	1.473
4½	.4131	.5508	.6885	.8262	.9639	1.1016	1.2393	1.377	1.6524
4¾	.4602	.6136	.7671	.9204	1.073	1.227	1.380	1.534	1.84

kind of fuel, such as coal, coke, wood, gas, or kerosene oil. They will pump from either a shallow or a deep well, but are best adapted to wells in which the surface of the water is within 20 ft. of the top of the well.

The best known hot-air engines are the Rider and Ericsson, which have been in successful operation for over twenty-six years.

These engines have capacities ranging from 150 to 3,500 gallons per hour and will deliver water from 50 to 350 ft. above the surface of water in the well, although the higher the water is raised the less will be the quantity delivered.

The cost of these engines with pump attached varies from $120 for the smallest size, having a capacity of 150 gals. per hour raised 50 ft., to $540 for the largest size, having a capacity of 3,500 gals. per hour raised 50 ft. The smaller size requires about 1 quart of kerosene or 3 lbs. of anthracite coal per hour.

Hot-air engines should be placed close to source of supply, and when the latter is a deep well the engine must be placed so that the pump-rod will be in a vertical line above the cylinder in the well, the operation of pumping being the same as that of the ordinary single-acting deep-well pump.

It is not practical to *draw* water more than 20 to 25 ft. (in height) with any form of suction pump, because of the difficulty of keeping the pipe, valve, and fittings absolutely air tight.

For further information, see catalogue of the Rider-Ericsson Engine Co.

Windmills.—In the country and on large suburban estates, windmills are extensively used for pumping water. Aside from the noise of operation, the only objection to the windmill (where it can be used) is the irregularity of its supply, but with a large storage tank this is not a serious objection when used for domestic purposes only. Prof. Thurston says, regarding wind mills: "In estimating the capacity, a working-day of eight hours is assumed, but the machine, when used for pumping, may actually do its work twenty-four hours a day for days, weeks, and even months together, whenever the wind is stiff enough to turn it. It costs for work done only one half or one third as much as steam, hot-air, or gas engines of similar power.

The wind mill operates the plunger in the well, the process of pumping being the same as that of the single-acting pumps described above.

The following table of capacity was prepared by Alfred R. Wolff, and is sufficiently accurate for all practical purposes:

CAPACITY OF THE WINDMILL.

Designation of Mill.	Velocity of Wind in Miles per Hr.	Revolutions of Wheel per Minute.	Gallons of Water Raised per Minute to an Elevation of						Equivalent Actual Useful H.P. Developed.
			25 Feet.	50 Feet.	75 Feet.	100 Feet.	150 Feet.	200 Feet.	
wheel Feet.									
8½	16	70 to 75	6.192	3.016	......	......	......	......	0.04
10	16	60 " 65	19.179	9.563	6.638	4.750	......	......	0.12
12	16	55 " 60	33.941	17.952	11.851	8.435	5.680	......	0.21
14	16	50 " 55	45.139	22.569	15.304	11.246	7.807	4.998	0.28
16	16	45 " 50	64.600	31.654	19.542	16.150	9.771	8.075	0.41
18	16	40 " 45	97.682	52.165	32.513	24.421	17.485	12.211	0.61
20	16	35 " 40	124.950	63.750	40.800	31.248	19.284	15.938	0.78
25	16	30 " 35	212.381	106.964	71.604	49.725	37.349	26.741	1.34

The horse-power of windmills of the best construction is proportional to the squares of their diameters and inversely as their velocities; for example, a 10-ft. mill in a 16-mile breeze

will develop 0.15 horse-power at 65 revolutions per minute; and with the same breeze:

A 20-ft.	mill	40 revolutions,	1	horse-power.
A 25-ft.	"	35 "	$1\frac{3}{4}$	" "
A 30-ft.	"	28 "	$3\frac{1}{2}$	" "
A 40-ft.	"	22 "	$7\frac{1}{2}$	" "
A 50-ft.	"	18 "	12	" "

The increase in power from increased velocity of the wind is equal to the square of its proportional velocity; as, for example, the 25-ft. mill rated above for a 16-mile wind will, with a 32-mile wind, have its horse-power increased to $4 \times 1\frac{3}{4} = 7$ horse-power.*

A windmill "will run and produce work in a 4-mile breeze."

Windmills have also been used successfully for the generating and storage of electricity for small lighting plants.†

Air-lift Process.—Compressed air is now being used to an increasing extent for raising water from artesian wells. The process in general consists of submerging a discharge pipe in a closed well, with a smaller pipe inside delivering compressed air into it at the bottom. The compressed air by its inherent expansive force lifts a column of mingled air and water which is conveyed to an open tank, to permit of the escape of the air. If desired the water may then be conveyed by gravity into a series of closed tanks, and forced by air pressure to different parts of a building, the only machinery required being an air-compressor and power for driving it.

The method of piping a well differs according to its general conditions and the quantity of water to be pumped. "No two wells are alike, and consequently the method of piping which might be applied to one would be unsuited to another."

Information as to the best method of piping any particular well may be obtained from the Ingersoll-Sergeant Drill Co.

Advantages of the Air-lift Process.—From two to six times as much water may be obtained from a given diameter of well as with any other known system, because there are no valves, cylinders, or rods to hinder the rapid discharge of water.

One air-compressor operates any number of wells, which may be any distance apart so as not to affect one another.

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* Kent, p. 497, quoted from the *Iron Age*.

† See Kent, p. 498.

There is nothing outside the engine-room to look after or wear out. Nothing but common pipe in the wells.

Water is cooled and purified by the thorough admixture and expansion of air; iron, sulphur, and gases are thrown off.

Sand or gravel does no harm.

The cost of raising 1,000 gallons of water by this method, including fuel, labor, oil, interest on cost of well, boiler, compressor, foundations, pipes, real estate, and erection, taxes, and fifteen per cent. for depreciation, runs from two and one-half cents down to one fifth of one cent, according to the size of the plant, height of lift, and other local conditions. With the average outfit of medium or small size, it is usually under one and one half cents.*

The air-lift process is now extensively used in iceworks, breweries, cold-storage houses, textile mills, dyeworks, etc., and a great variety of industrial plants, and for the water-supply of quite a number of the smaller cities.

In Newark, N. J., pumps of this type are at work having a total capacity of 1,000,000 gallons daily, lifting water from three 8-in. artesian wells. (Kent.)

Horse-power Required to Raise Water to Different Heights.

The power required to raise a certain quantity of water to a certain height varies directly with the quantity to be raised, and also the height.

For instance, it requires twice as much power to raise 200 gallons per minute 10 ft. high as it does to raise 100 gallons to the same height and in the same time; and to raise 100 gallons 20 ft. high requires twice as much power as it does to raise 100 gallons 10 ft. high.

To find the theoretical horse-power necessary to elevate water to a given height, multiply the number of gallons per minute by 8.35, weight of one gallon, and this result by the total number of feet the water is raised (that is, from the surface of the water to the highest point to which the water is raised), and the result gives the power in foot-pounds; divide by 33,000, and the quotient is the horse-power. To the theoretical power a liberal allowance must be made for the inefficiency of the pump.

* Ingersoll-Sergeant Drill Co.

For a cylinder pump add 75 to 100 per cent.

To the actual height to which the water is to be raised add the friction loss in feet, as given in Table F, when the discharge is to be piped any distance.

EXAMPLE.—Find the theoretical horse-power required to raise 100 gallons per minute 120 ft. high, through a 3-in. pipe, 200 ft. long.

Ans. From Table F, the friction head for 100 gallons per minute in 3-in. pipe, 100 ft. long, is 1.31×2.3 or 3 ft. For 200 ft. it will be 6 ft., which added to 120 gives 126 ft. for the height. Then theoretical horse-power $= \dfrac{100 \times 8.35 \times 126}{33,000}$ $= 3.2$ H.P. The actual horse-power required will probably vary from 5 to 6, according to the efficiency of the pump.

The mistake of using too small a discharge pipe can easily be seen from Table F.

For instance, if one attempted to force 100 gallons per minute through 100 ft. of 2-in. pipe, the back pressure would be equivalent to raising the water 22 ft. high. The fuel used would be correspondingly increased. Right-angle turns are to be avoided, as the friction is very materially increased, being practically equal to the friction of 25 ft. of straight pipe.

Fire Streams.

The following is an extract from a paper read by Mr. John R. Freeman at a meeting of the New England Waterworks Association, entitled "Some Experiments and Practical Tables Relating to Fire Streams."

"When unlined linen hose is used the friction or pressure loss is from 8 to 60 per cent., increasing with the pressure. This kind of hose is best for inside use in short lengths. Mill hose is better than unlined linen hose for long lengths, but ordinarily the best quality of smooth rubber-lined hose is superior to the mill hose, having less frictional resistance.

"The ring nozzle is inferior to the smooth nozzle and actually delivers less water than the smooth nozzle. For instance, the $\frac{7}{8}''$ ring nozzle discharges the same quantity of water as a $\frac{3}{4}''$ smooth, and a $1''$ ring nozzle the same as a $\frac{7}{8}''$ smooth.

"Two hundred and fifty gallons per minute is a good standard fire stream at 80 lbs. pressure at the hydrant. 100 lbs. pressure

should not be exceeded except for very high buildings or lengths of hose exceeding 300 ft."

TABLE OF EFFECTIVE FIRE STREAMS,

Using 100 ft. of $2\frac{1}{2}''$ ordinary best quality rubber-lined hose between nozzle and hydrant or pump.

Smooth Nozzle, Size. . . .	¾-inch.					⅞-inch.				
Pressure at hydrant, lbs. .	32	54	65	75	86	34	5[illegible]	69	80	91
Pressure at nozzle, lbs. . .	30	50	60	70	80	30	5[illegible]	60	70	80
Vertical height, feet.	48	67	72	76	79	49	71	77	81	85
Horizontal distance, feet	37	50	54	68	62	42	55	61	66	70
Gals. discharged per min.	90	116	127	137	147	123	159	[illegible]74	188	201

Smooth Nozzle, Size. . . .	1-inch.					1⅛-inch.				
Pressure at hydrant, lbs .	37	62	75	87	100	42	70	84	98	112
Pressure at nozzle, lbs. . .	30	50	60	70	80	30	50	60	70	80
Vertical height, feet.	51	73	79	85	89	52	75	83	88	92
Horizontal distance, feet	47	61	67	72	76	50	66	72	77	81
Gals. discharged per min.	161	208	228	246	263	206	266	291	314	336

Notes on the Construction of Cylindrical Wooden Tanks.*

Material should be either cedar, cypress, or white pine, free from imperfections and thoroughly air-dry. Where exposed to freezing, Michigan pine free from sapwood is generally considered the most durable.

Staves and bottom to be made of $2\frac{1}{2}$-inch (dressed to about $2\frac{1}{4}$-inch) stock for tanks 12 ft. and not exceeding 16 ft. diameter or 16 ft. deep. For larger tanks 3-in. (dressed to about $2\frac{3}{4}$-in.) stock to be used.

Staves to be connected about one third the distance from the top by a $\frac{5}{8}$-inch dowel to hold in position during erection.

The bottom planks to be dressed four sides, and the edges of each plank to be bored with holes not over 3 feet apart for $\frac{5}{8}$-inch dowels.

Taper.—The batter to each side should not be less than $\frac{1}{4}$ in. nor more than $\frac{1}{2}$ in. per foot.

* These notes have been condensed from specifications published by the Inspection Department of the Factory Mutual Fire Insurance Co., 31 Milk Street, Boston; a most excellent pamphlet.

Hoops.—All to be of *round* wrought iron or mild steel of good quality. Wrought iron is preferable because it does not rust so easily as steel.

There are to be no welds in any of the hoops. Where more than one length of iron is necessary, lugs are to be used to make the joints; and when more than one piece is necessary the several pieces constituting one hoop should be tied together in preparing for shipment.

Hoops to be chosen of such a size and spacing that the stress in no hoop will exceed 12,500 lbs. per square inch when computed from the area at root of thread.

On account of the swelling of the bottom planks, the hoops near the bottom may be subjected to a strain greater than that due to the water pressure alone; therefore additional hoops should be provided. For tanks up to 20 ft. in diameter, one hoop of the size used next above it should be placed around the bottom opposite the croze and not counted upon as withstanding any water pressure. For tanks 20 ft. or more in diameter, two hoops, as above, should be used.

Hoops with "upset" ends must not be used. The top hoop to be placed within 2 ins. of the top of staves, so that overflow pipe may be inserted as high as possible. Hoops to be so placed that the lugs will not come in a vertical line. No hoop to be less than $\frac{3}{4}$ in. diameter. All to be cleaned of mill-scale and rust and painted one coat red lead, lampblack, and boiled oil before erecting.

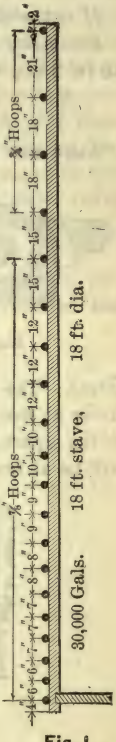


Fig. 1

[NOTE.—The strength of a tank depends chiefly on its hoops. Round hoops are specified because they do not rust as quickly; a slight amount of rust does not have the same weakening effect as on a flat hoop, and round hoops are not likely to burst when the tank swells, as they will sink into the wood.]

Spacing of Hoops.—The hoops to be spaced so that each hoop will have the same stress per square inch, and no space to be greater than 21 ins.

To meet this requirement the hoops must be spaced quite close together at the bottom, the space between hoops gradually increasing towards the top. Fig. 1 shows the proper spacing of hoops for a tank 18 ft. diameter with 18-ft. staves. The spacing for seven other sizes of tanks is given in the pamphlet referred to. It may be computed by the following formula:

$$\text{Spacing of hoop in inches} = \frac{\text{Strength}}{2.6 \times \text{diameter in feet} \times H}.$$

For strength of a $\frac{3}{4}$-inch rod use 3,750; of a $\frac{7}{8}$-in. rod, 5,250; of a 1-in. rod, 6,875; and of a $1\frac{1}{8}$-in. rod, 8,625.

H is the distance from top of water to centre of hoop in feet.

EXAMPLE.—How far apart should 1-in. hoops be placed, at 15 ft. 2 ins. from top of tank, on a tank 20 ft. diameter?

$$Ans. \quad \frac{6,875}{2.6 \times 20 \times 15} = 8\tfrac{3}{4} \text{ ins.}$$

Lugs are to be as strong as the hoops. A lug similar to Fig. 2 is simple and fulfils the requirement for strength. Malleable lugs are preferable.

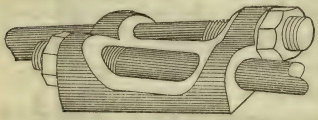


Fig. 2
Lug for Hoops.

Support.—The weight of the tank should be supported entirely from its bottom; and *in no event should any weight come on the bottom of the staves.* The planks upon which the tank bottom rests should cover at least one fifth the area of the bottom and be not over 18 ins. apart, and of such thickness that the bottom of the staves will be at least an inch from the floor (see Fig. 3).

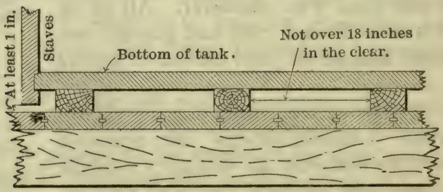


Fig. 3
Support for Bottom of Tank.

Discharge Pipe will preferably leave the bottom of the tank at its centre and extend up inside of the tank 4 ins., to allow for sediment collecting in the bottom of the tank.

The Overflow Pipe should be placed as near the top of the

tank as possible, discharging either through side or bottom, as may be desired. An overflow is much to be preferred to a telltale, as the latter is liable to get out of order.

Heating.—Tanks of moderate size need to be provided with some means to prevent freezing.

When a tank is in an enclosed room, as in a mill tower, the best method is to keep the room wa m by a coil of steam-pipe with a return to the boiler-room. A covered tank out of doors may often be similarly heated by placing the steam-pipe in the bottom of the tank.

With a tank located on a high trestle, or at a distance from the steam-supply, it is often impracticable to arrange a return pipe. In this case steam may be blown directly into the water in the tank. A 1-inch pipe is generally sufficient for this purpose. It should be carried to the top of the tank and there bend over and dip downwards, so that its outlet is about 1 foot below the high-water line. A check-valve is to be placed in this steam-pipe, near its point of discharge, to prevent water being drawn back by siphon action when the steam is shut off.

Frost-proofing for Pipes — The discharge pipe from a tank on a trestle, or one elevated above the roof, must be protected from freezing. The most common practice is to enclose

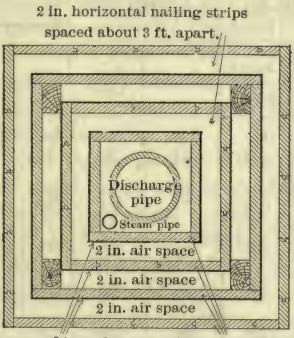


Fig. 4
Method of Frost-proofing Pipes.

the pipe in a double, triple, or quadruple box made of boards and tarred paper as shown by Fig. 4. If steam is supplied to the tank, the steam-pipe is carried inside the box.

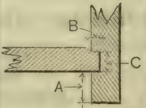
In New England, New York, and Canada the quadruple boxing is generally used, whereas in the milder regions to the south triple or double boxing is used.

The boxing should always be carried down into the ground below the frost-line, and a good tight joint made at the underside of the tank.

Covers.—For economy in heating and to prevent birds, leaves, etc., from getting into the water, all out-of-door tanks should be covered. A double cover is recommended consisting of a tight flat cover made of matched boards supported by joists which span the top of the tank, and above this a shingled, conical roof. To prevent the covering from being blown off, it should be firmly fastened to the top of the tank by straps of iron.

In order to keep out the wind particular attention should be given to making a tight joint where the roof rests on the top of the staves.

DIMENSIONS OF TANKS OF STANDARD SIZES.

Approximate Net Capacity.	Size (Outside Dimensions).		Thickness of Lumber after being Machined.					Hoops.	
	Average Diameter.	Length of Stave.	Staves.	Bottom.	A	B	C	Number of	Size.
Gallons.	Ft. Ins.	Ft.	Ins.	Ins.	Ins.	In.	Ins.		Ins.
10,000	13 4	12	$2\frac{1}{4}$	$2\frac{1}{4}$	$3\frac{1}{2}$	$\frac{5}{8}$	$2\frac{1}{8}$	11	$\frac{3}{4}$
15,000	14 6	14	$2\frac{1}{4}$	$2\frac{1}{4}$	$3\frac{1}{2}$	$\frac{5}{8}$	$2\frac{1}{8}$	14	$\frac{3}{4}$
20,000	15 6	16	$2\frac{1}{4}$	$2\frac{1}{4}$	$3\frac{1}{2}$	$\frac{5}{8}$	$2\frac{1}{8}$	5 11	$\frac{3}{4}$ $\frac{7}{8}$
25,000	17 6	16	$2\frac{3}{4}$	$2\frac{3}{4}$	$3\frac{1}{2}$	$\frac{3}{4}$	$2\frac{5}{8}$	4 12	$\frac{3}{4}$ $\frac{7}{8}$
30,000	18 0	18	$2\frac{3}{4}$	$2\frac{3}{4}$	$3\frac{1}{2}$	$\frac{3}{4}$	$2\frac{5}{8}$	4 16	$\frac{3}{4}$ $\frac{7}{8}$
50,000	22 0	20	$2\frac{3}{4}$	$2\frac{3}{4}$	$3\frac{1}{2}$	$\frac{3}{4}$	$2\frac{5}{8}$	4 19	$\frac{7}{8}$ $\frac{7}{8}$
75,000	24 6	24	$2\frac{3}{4}$	$2\frac{3}{4}$	$3\frac{1}{2}$	$\frac{3}{4}$	$2\frac{5}{8}$	4 6 21	$\frac{7}{8}$ 1 $1\frac{1}{8}$
100,000	28 6	24	$2\frac{3}{4}$	$2\frac{3}{4}$	$3\frac{1}{2}$	$\frac{3}{4}$	$2\frac{5}{8}$	5 29	1 $1\frac{1}{8}$

Scuttles should be arranged in both the conical and flat covers to give access to the inside of the tank and a substantial, permanent ladder erected to give easy access to the top of the tank.

Notes on Steel Tanks.*

Steel tanks of sizes commonly used for fire protection cost from 40 to 100 per cent. more than do wooden tanks. The additional cost for large tanks is relatively less than for small tanks. A steel tank of about 40,000 gallons capacity or over can be erected on a steel trestle at about the same cost as a wooden tank, since a saving can be made in the cost of supports by making a hemispherical or conical bottom to the steel tank and supporting the tank directly on the legs of the trestle, thus saving the expense of horizontal supporting beams.

A steel tank is superior to a wooden tank in (1) that it will last for an indefinite time if *kept thoroughly painted* inside and out, whereas a wooden tank will have to be replaced in from twelve to thirty years (usually about fifteen years); (2) that it will be absolutely tight when once well erected and properly cared for, whereas a wooden tank will shrink and leak if the water gets low; (3) that it will not be at all likely to burst suddenly (if originally correctly designed) even if painting is neglected, for experience shows that a few spots will first rust through and thus show the weak condition by small leaks, whereas a wooden tank, if neglected, may burst its hoops suddenly and cause serious damage.

The objections to steel tanks are: (1) They require skilled boiler-makers to erect them, thus adding considerable to the cost when erected at a distance from a boiler-shop; (2) they are more difficult to protect against freezing; (3) they give more trouble by "sweating" when placed in a mill tower; (4) they deteriorate rapidly if painting is neglected.

*** Inspection Department of the Factory Mutual Insurance Co., Boston.**

CONTENTS IN CUBIC FEET AND U. S. GALLONS OF PIPES AND CYLINDERS OF VARIOUS DIAMETERS AND ONE FOOT IN LENGTH.

1 gallon = 231 cubic inches. 1 cubic foot = 7.4805 gallons.

Diameter, in Inches.*	For 1 Foot in Length.		Diameter, in Inches.	For 1 Foot in Length.		Diameter, in Inches.	For 1 Foot in Length.	
	Cu. Ft., also Area in Sq. Ft.	U. S. Gals., 231 Cu.In.		Cu. Ft., also Area in Sq. Ft.	U. S. Gals., 231 Cu. In.		Cu. Ft., also Area in Sq. Ft.	U. S. Gals., 231 Cu. In.
1/4	.0003	.0025	6 3/4	.2485	1.859	19	1.969	14.73
5/16	.0005	.004	7	.2673	1.999	19 1/2	2.074	15.51
3/8	.0008	.0057	7 1/4	.2867	2.145	20	2.182	16 32
7/16	.001	.0078	7 1/2	.3068	2.295	20 1/2	2.292	17.15
1/2	.0014	.0102	7 3/4	.3276	2.45	21	2.405	17.99
9/16	.0017	.0129	8	.3491	2.611	21 1/2	2.521	18.86
5/8	.0021	.0159	8 1/4	.3712	2.777	22	2.640	19.75
11/16	.0026	.0193	8 1/2	.3941	2.948	22 1/2	2.761	20.66
3/4	.0031	.0230	8 3/4	.4176	3.125	23	2.885	21.58
13/16	.0036	.0269	9	.4418	3.305	23 1/2	3.012	22.53
7/8	.0042	.0312	9 1/4	.4667	3.491	24	3.142	23.50
15/16	.0048	.0359	9 1/2	.4922	3.682	25	3.409	25.50
1	.0055	.0408	9 3/4	.5185	3.879	26	3.687	27.58
1 1/4	.0085	.0638	10	.5454	4.08	27	3.976	29.74
1 1/2	.0123	.0918	10 1/4	.5730	4.286	28	4.276	31.99
1 3/4	.0167	.1249	10 1/2	.6013	4.498	29	4.587	34.31
2	.0218	.1632	10 3/4	.6303	4.715	30	4.909	36.72
2 1/4	.0276	.2066	11	.66	4.937	31	5.241	39.21
2 1/2	.0341	.2550	11 1/4	.6903	5.164	32	5.585	41.78
2 3/4	.0412	.3085	11 1/2	.7213	5.396	33	5.940	44.43
3	.0491	.3672	11 3/4	.7530	5.633	34	6.305	47.16
3 1/4	.0576	.4309	12	.7854	5.875	35	6.681	49.98
3 1/2	.0668	.4998	12 1/2	.8522	6.375	36	7.069	52.88
3 3/4	.0767	.5738	13	.9218	6.895	37	7.467	55.86
4	.0873	.6528	13 1/2	.994	7.436	38	7.876	58.92
4 1/4	.0985	.7369	14	1.069	7.997	39	8.296	62.06
4 1/2	.1134	.8263	14 1/2	1.147	8.578	40	8.727	65.28
4 3/4	.1231	.9206	15	1.227	9.180	41	9.168	68.58
5	.1364	1.020	15 1/2	1.310	9.801	42	9.621	71.97
5 1/4	.1503	1.125	16	1.396	10.44	43	10.085	75.44
5 1/2	.1650	1.234	16 1/2	1.485	11.11	44	10.559	78.99
5 3/4	.1803	1.349	17	1.576	11.79	45	11.045	82.62
6	.1963	1.469	17 1/2	1.670	12.49	46	11.541	86.33
6 1/4	.2131	1.594	18	1.768	13.22	47	12.048	90.13
6 1/2	.2304	1.724	18 1/2	1.867	13.96	48	12.566	94.00

* Actual.

To find the capacity of pipes greater than those given, look in the table for a pipe of one half the given size and multiply its capacity by 4, or one of one third its size and multiply its capacity by 9, etc.

The find the *weight* of water in any of the given sizes multiply the capacity in cubic feet by the weight of a cubic foot of water at the temperature of the water in the pipe (see page 1112).

To find the capacity of a cylinder in U. S. gallons multiply the length by the square of the diameter and by 0.0034.

CYLINDRICAL VESSELS, TANKS, CISTERNS, ETC.

Diameter in feet and inches, area in square feet, and U. S. gallons capacity for one foot in depth.

1 gallon = 231 cubic inches = 0.1337 cubic foot.

Diam.	Area.	Gals.	Diam.	Area.	Gals.	Diam.	Area.	Gals.
Ft. In.	Sq. Ft. *	1-Foot Depth.	Ft. In.	Sq. Ft. *	1-Foot Depth.	Ft. In.	Sq. Ft. *	1-Foot Depth.
1	.785	5.87	5 8	25.22	188.66	19	283.53	2120.9
1 1	.922	6.89	5 9	25.97	194.25	19 3	291.04	2177.1
1 2	1.069	8.00	5 10	26.73	199.92	19 6	298.65	2234.0
1 3	1.227	9.18	5 11	27.49	205.67	19 9	306.35	2291.7
1 4	1.396	10.44	6	28.27	211.51	20	314.16	2350.1
1 5	1.576	11.79	6 3	30.68	229.50	20 3	322.06	2409.2
1 6	1.767	13.22	6 6	33.18	248.23	20 6	330.06	2469.1
1 7	1.969	14.73	6 9	35.78	267.69	20 9	338.16	2529.6
1 8	2.182	16.32	7	38.48	287.88	21	346.36	2591.0
1 9	2.405	17.99	7 3	41.28	308.81	21 3	354.66	2653.0
1 10	2.640	19.75	7 6	44.18	330.48	21 6	363.05	2715.8
1 11	2.885	21.58	7 9	47.17	352.88	21 9	371.54	2779.3
2	3.142	23.50	8	50.27	376.01	22	380.13	2843.6
2 1	3.409	25.50	8 3	53.46	399.88	22 3	388.82	2908.6
2 2	3.687	27.58	8 6	56.75	424.48	22 6	397.61	2974.3
2 3	3.976	29.74	8 9	60.13	449.82	22 9	406.49	3040.8
2 4	4.276	31.99	9	63.62	475.89	23	415.48	3108.0
2 5	4.587	34.31	9 3	67.20	502.70	23 3	424.56	3175.9
2 6	4.909	36.72	9 6	70.88	530.24	23 6	433.74	3244.6
2 7	5.241	39.21	9 9	74.66	558.51	23 9	443.01	3314.0
2 8	5.585	41.78	10	78.54	587.52	24	452.39	3384.1
2 9	5.940	44.43	10 3	82.52	617.26	24 3	461.86	3455.0
2 10	6.305	47.16	10 6	86.59	647.74	24 6	471.44	3526.6
2 11	6.681	49.98	10 9	90.76	678.95	24 9	481.11	3598.9
3	7.069	52.88	11	95.03	710.90	25	490.87	3672.0
3 1	7.467	55.86	11 3	99.40	743.58	25 3	500.74	3745.8
3 2	7.876	58.92	11 6	103.87	776.99	25 6	510.71	3820.3
3 3	8.296	62.06	11 9	108.43	811.14	25 9	520.77	3895.6
3 4	8.727	65.28	12	113.10	846.03	26	530.93	3971.6
3 5	9.168	68.58	12 3	117.86	881.65	26 3	541.19	4048.4
3 6	9.621	71.97	12 6	122.72	918.00	26 6	551.55	4125.9
3 7	10.085	75.44	12 9	127.68	955.09	26 9	562.00	4204.1
3 8	10.559	78.99	13	132.73	992.91	27	572.56	4283.0
3 9	11.045	82.62	13 3	137.89	1031.5	27 3	588.21	4362.7
3 10	11.541	86.33	13 6	143.14	1070.8	27 6	593.96	4443.1
3 11	12.048	90.13	13 9	148.49	1110.8	27 9	604.81	4524.3
4	12.566	94.00	14	153.94	1151.5	28	615.75	4606.2
4 1	13.095	97.96	14 3	159.48	1193.0	28 3	626.80	4688.8
4 2	13.635	102.00	14 6	165.13	1235.3	28 6	637.94	4772.1
4 3	14.186	106.12	14 9	170.87	1278.2	28 9	649.18	4856.2
4 4	14.748	110.32	15	176.71	1321.9	29	660.52	4941.0
4 5	15.321	114.61	15 3	182.65	1366.4	29 3	671.96	5026.6
4 6	15.90	118.97	15 6	188.69	1411.5	29 6	683.49	5112.9
4 7	16.50	123.42	15 9	194.83	1457.4	29 9	695.13	5199.9
4 8	17.10	127.95	16	201.06	1504.1	30	706.86	5287.7
4 9	17.72	132.56	16 3	207.39	1551.4	30 3	718.69	5376.2
4 10	18.35	137.25	16 6	213.82	1599.5	30 6	730.62	5465.4
4 11	18.99	142.02	16 9	220.35	1648.4	30 9	742.64	5555.4
5	19.63	146.88	17	226.98	1697.9	31	754.77	5646.1
5 1	20.29	151.82	17 3	233.71	1748.2	31 3	766.99	5737.5
5 2	20.97	156.83	17 6	240.53	1799.3	31 6	779.31	5829.7
5 3	21.65	161.93	17 9	247.45	1851.1	31 9	791.73	5922.6
5 4	22.34	167.12	18	254.47	1903.6	32	804.25	6016.2
5 5	23.04	172.38	18 3	261.59	1956.8	32 3	816.86	6110.6
5 6	23.76	177.72	18 6	268.80	2010.8	32 6	829.58	6205.7
5 7	24.48	183.15	18 9	276.12	2065.5	32 9	842.39	6301.5

* Also cubic feet for 1 foot in depth.

CAPACITY OF CISTERNS AND TANKS.

Number of Barrels (31½ Gals.) in Cisterns and Tanks.

Depth, in Feet.	Diameter, in Feet.								
	5	6	7	8	9	10	11	12	13
5	23.3	33.6	45.7	59.7	75.5	93.2	112.8	134.3	157.6
6	28.0	40.3	54.8	71.7	90.6	111.9	135.4	161.1	189.1
7	32.7	47.0	64.0	83.6	105.7	130.6	158.0	188.0	220.6
8	37.3	53.7	73.1	95.5	120.9	149.2	180.5	214.8	252.1
9	42.0	60.4	82.2	107.4	136.0	167.9	203.1	241.7	283.7
10	46.7	67.1	91.4	119.4	151.1	186.5	225.7	268.6	315.2
11	51.3	73.9	100.5	131.3	166.2	205.1	248.2	295.4	346.7
12	56.0	80.6	109.7	143.2	181.3	223.8	270.8	322.3	378.2
13	60.7	87.3	118.8	155.2	196.4	242.4	293.4	349.1	409.7
14	65.3	94.0	127.9	167.1	211.5	261.1	315.9	376.0	441.3
15	70.0	100.7	137.1	179.0	226.6	289.8	338.5	402.8	472.8
16	74.7	107.4	146.2	191.0	241.7	298.4	361.1	429.7	504.3
17	79.3	114.1	155.4	202.9	256.8	317.0	383.6	456.6	535.8
18	84.0	120.9	164.5	214.8	272.0	335.7	406.2	483.4	567.3
19	88.7	127.6	173.6	226.8	287.0	354.3	428.8	510.3	598.0
20	93.3	134.3	182.8	238.7	302.1	373.0	451.3	537.1	630.4

Depth, in Feet.	Diameter, in Feet.								
	14	15	16	17	18	19	20	21	22
5	182.8	209.8	238.7	269.5	302.1	336.6	373.0	411.2	451.3
6	219.3	251.8	286.5	323.4	362.6	404.0	447.6	493.5	541.6
7	255.9	293.7	334.2	377.3	423.0	471.3	522.2	575.7	631.9
8	292.4	335.7	382.0	431.2	483.4	538.6	596.8	658.0	722.1
9	329.0	377.7	429.7	485.1	543.8	605.9	671.4	740.2	812.4
10	365.5	419.6	477.4	539.0	604.3	673.3	746.0	822.5	902.7
11	402.1	461.6	525.2	592.9	667.7	740.6	820.6	904.7	992.9
12	438.6	503.5	572.9	646.8	725.1	807.9	895.2	987.0	1083.2
13	475.2	545.5	620.7	700.7	785.5	875.2	969.8	1069.2	1173.5
14	511.8	587.5	668.2	754.6	846.0	942.6	1044.4	1151.5	1263.7
15	548.3	629.4	716.2	808.5	906.4	1009.9	1119.0	1233.7	1354.0
16	584.9	671.4	773.9	862.4	966.8	1077.2	1193.6	1315.9	1444.3
17	621.4	713.4	811.6	916.3	1027.2	1044.6	1268.2	1398.2	1534.5
18	658.0	755.3	859.4	970.2	1087.7	1211.9	1342.8	1480.4	1624.8
19	694.5	797.3	907.1	1024.1	1148.1	1279.2	1417.4	1562.7	1715.1
20	731.1	839.3	954.9	1078.0	1208.5	1346.5	1492.0	1644.9	1805.3

Depth, in Feet.	Diameter, in Feet.							
	23	24	25	26	27	28	29	30
5	493.3	537.1	582.8	630.4	679.8	731.1	784.2	839.3
6	592.0	644.5	699.4	756.5	815.8	877.3	941.1	1007.1
7	690.6	752.0	815.9	882.5	951.7	1023.5	1097.9	1175.0
8	789.3	859.4	932.5	1008.6	1087.7	1169.7	1254.8	1342.8
9	887.9	966.8	1049.1	1134.7	1223.6	1316.0	1411.6	1510.7
10	986.6	1074.2	1165.6	1260.8	1359.6	1462.2	1568.2	1678.5
11	1085.2	1181.7	1282.2	1386.8	1495.6	1608.7	1723.0	1846.4
12	1183.9	1289.1	1398.7	1512.9	1631.5	1754.6	1882.2	2014.2
13	1282.6	1396.5	1515.3	1639.0	1767.5	1900.8	2039.0	2182.0
14	1381.2	1503.9	1631.9	1765.1	1903.4	2047.1	2195.9	2343.9
15	1479.9	1611.4	1748.4	1891.1	2039.4	2193.3	2352.7	2517.8
16	1578.5	1718.8	1865.0	2017.2	2175.4	2339.5	2509.6	2685.6
17	1677.2	1826.2	1981.6	2143.3	2311.3	2485.7	2666.4	2853.5
18	1775.9	1933.6	2098.1	2269.4	2447.3	2631.9	2823.3	3021.3
19	1874.5	2041.1	2214.7	2395.4	2583.2	2778.1	2980.1	3189.2
20	1973.2	2148.5	2321.2	2521.5	2719.2	2924.4	3137.0	3357.0

For tanks that are tapering, measure the diameter four tenths from large end.

NUMBER OF U. S. GALLONS IN RECTANGULAR TANKS

For One Foot in Depth.

1 cu. ft. = 7.4805 gallons.

Width, in Feet.	Length of Tank, in Feet.										
	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
2	29.92	37.40	44.88	52.36	59.84	67.32	74.81	82.29	89.77	97.25	104.73
2.5	.....	46.75	56.10	65.45	74.80	84.16	93.51	102.8	112.21	121.56	130.91
3	.....	.....	67.32	78.54	89.77	100.99	112.21	123.43	134.65	145.87	157.09
3.5	.....	.....	.....	91.64	104.73	117.82	130.91	144.00	157.09	170.18	183.27
4	.....	.....	.....	.....	119.69	134.65	149.61	164.57	179.53	194.49	209.45
4.5	.....	.....	.....	.....	.....	151.48	168.31	185.14	201.97	218.80	235.63
5	.....	.....	.....	.....	.....	.....	187.01	205.71	224.41	243.11	261.82
5.5	.....	.....	.....	.....	.....	.....	.....	226.28	246.86	267.43	288.00
6	.....	.....	.....	.....	.....	.....	.....	.....	269.30	291.74	314.18
6.5	.....	.....	.....	.....	.....	.....	.....	.....	.....	316.05	340.36
7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	366.54

Width, in Feet.	Length of Tank, in Feet.									
	7.5	8	8.5	9	9.5	10	10.5	11	11.5	12
2	112.21	119.69	127.17	134.65	142.13	149.61	157.09	164.57	172.05	179.53
2.5	140.26	149.61	158.96	168.31	177.66	187.01	196.36	205.71	215.06	224 41
3	168.31	179.53	190.75	202.97	213.19	224.41	235.63	246.86	258.07	269.30
3.5	196.36	209.45	222.54	235.63	248.73	261.82	274.90	288.00	301.09	314.18
4	224.41	239.37	254.34	269.30	284.26	299.22	314.18	329.14	344.10	359.06
4.5	252.47	269.30	286.13	302.96	319.79	336.62	353.45	370.28	387.11	403.94
5	280.52	299.22	317.92	336.62	355.32	374.03	392.72	411.43	430.13	448.83
5.5	308.57	329.14	349.71	370.28	390.85	411.43	432.00	452.57	473.14	493.71
6	336.62	359.06	381.50	403.94	426.39	448.83	471.27	493.71	516.15	538.59
6.5	364 67	388.98	413 30	437.60	461.92	486.23	510.54	534.85	550.16	583.47
7	392.72	418.91	445.09	471.27	497.45	523.64	549.81	575.00	602.18	628.36
7.5	420.78	448.83	476.88	504.93	532.98	561.04	589.08	617.14	645.19	673.24
8	.....	478.75	508.67	538.59	568.51	598.44	628.36	658.28	688.20	718.12
8.5	......	......	540.46	572.25	604.05	635.84	667.63	690.42	731.21	703.00
9	......	......	......	605.92	639.58	673.25	706.90	740.56	774.23	807.89
9.5	......	......	......	......	675.11	710.65	746.17	781.71	817.24	852.77
10	......	......	......	......	......	748.05	785.45	822.86	860.26	897.66
10.5	......	......	......	......	......	......	824.73	864.00	903.26	942.56
11	......	......	......	......	......	......	......	905.14	946.27	987.43
11.5	......	......	......	......	......	......	......	......	989.29	1032.3
12	......	......	......	......	......	......	......	......	......	1077.2

To find weight of water in pounds at 62° F. multiply number of gallons by $8\frac{1}{3}$.

Example.—To find number of gallons in a rectangular tank that is 7.5 ft. by 10 ft., the water being 4 ft. deep: Look in extreme left-hand column for 7.5 and opposite to this in column headed "10" read 561.04, which being multiplied by 4, the depth of water in the tank, gives 2244.2, the number of gallons required.

Plumbing.

The water-supply of buildings, including the apparatus for heating water, the system of drainage and sewage, and the various fixtures connected therewith, are installed by the plumber, usually in accordance with specifications prepared by the architect and subject to municipal regulations. An efficient and safe system of plumbing is a matter of vital importance. The following

EXTRACTS FROM THE RULES AND REGULATIONS OF THE DEPARTMENT OF BUILDINGS OF THE CITY OF NEW YORK

may be used as a reliable guide in any locality.

Definition of Terms.

12.* The term "private sewer" is applied to main sewers that are not constructed by and under the supervision of the Department of Sewers.

13. The term "house sewer" is applied to that part of the main drain or sewer extending from a point 2 ft. outside of the outer wall of building vault or area to its connection with public sewer, private sewer, or cesspool.

14. The term "house drain" is applied to that part of the main horizontal drain and its branches inside the walls of the building vault or area and extending to and connecting with the house sewer.

15. The term "soil-pipe" is applied to any vertical line of pipe extending through roof, receiving the discharge of one or more water-closets with or without other fixtures.

16. The term "waste-pipe" is applied to any pipe, extending through roof, receiving the discharge from any fixtures except water-closets.

17. The term "vent-pipe" is applied to any special pipe provided to ventilate the system of piping and to prevent trap siphonage and back pressure.

Materials and Workmanship.

Soil- and Vent-pipe.—19. All cast-iron pipes and fittings must be uncoated, sound, cylindrical, and smooth, free from cracks, sand holes, and other defects, and of uniform thickness and of the grade known in commerce as "extra heavy."

* Paragraph numbers are the same as those in the Official Regulations. Missing numbers show where paragraphs have been omitted.

20.* Pipe, including the hub, shall weigh not less than the following average weights per lineal foot:

Diameters.	Weights per Lineal Foot.
2 inches	5½ pounds
3 "	9½ "
4 "	13 "
5 "	17 "
6 "	20 "
7 "	27 "
8 "	33½ "
10 "	45 "
12 "	54 "

22. All joints must be made with picked oakum and molten lead and be made gas-tight. Twelve (12) oz. of fine, soft pig lead must be used at each joint for each inch in the diameter of the pipe.

24. Wrought-iron and steel pipes must be galvanized, and each length must have the weight and maker's name stamped on it.

30. All brass pipe for soil-, waste-, and vent-pipes and solder nipples must be thoroughly annealed, seamless, drawn, brass tubing of standard iron-pipe gauge.

Lead Waste-pipes.—38. The use of lead pipes is restricted to the short branches of the soil- and waste-pipes, bends and traps, and roof connections of inside leaders. "Short branches" of lead pipe shall be construed to mean not more than

5 feet of 1½ inch pipe
4 " " 2 " "
2 " " 3 " "
2 " " 4 " "

39. All connections between lead pipes and between lead and brass or copper pipes must be made by means of "wiped" solder joint.

40. All lead waste, soil, vent, and flush pipes must be of the best quality, known in commerce as "D," and of not less than the following weights per lineal foot:

* See foot-note on page 1262.

Diameters.	Weights per Lineal Foot.
$1\frac{1}{4}$ inches (for flush-pipes only)................	$2\frac{1}{2}$ pounds
$1\frac{1}{2}$ "	3 "
2 "	4 "
3 "	6 "
4 and $4\frac{1}{2}$ inches........................	8 "

41. All lead traps and bends must be of the same weights and thicknesses as their corresponding pipe branches. Sheet lead for roof flashings must be 6-lb. lead and must extend not less than 6 ins. from the pipe, and the joint made water-tight.

42. Copper tubing when used for inside leader roof connections must be seamless-drawn tubing not less than 22 gauge, and when used for roof flashings must be not less than 18 gauge.

Yard, Area, and Other Drains.

57. All yards, areas, and courts must be drained.

58. Tenement-houses and lodging-houses must have their yards, areas, and courts drained into the sewer.

59. These drains, when sewer connected, must have connections not less than 3 ins. in diameter. They should be controlled by one trap—the leader trap if possible.

Leaders.

63. All buildings shall be kept provided with proper metallic leaders for conducting water from the roofs in such manner as shall protect the walls and foundations of said buildings from injury. In no case shall the water from said leaders be allowed to flow upon the sidewalk, but the same shall be conducted by pipe or pipes to the sewer. If there be no sewer in the street upon which such buildings front, then the water from said leaders shall be conducted by proper pipe or pipes below the surface of the sidewalk to the street gutter.

64. Inside leaders must be made of cast iron, wrought iron, or steel, with roof connections made gas- and water-tight by means of a heavy lead or copper-drawn tubing wiped or soldered to a brass ferrule or nipple calked or screwed into the pipe.

65. Outside leaders may be of sheet metal, but they must connect with the house drain by means of a cast-iron pipe extending vertically 5 ft. above the grade level.

66. Leaders must be trapped with cast-iron running traps so placed as to prevent freezing.

67. Rain-water leaders must not be used as soil-, waste-, or vent-pipes, nor shall any such pipe be used as a leader.

The House Sewer, House Drain, House Trap, and Fresh-air Inlet.

72. The house drain must properly connect with the house sewer at a point 2 ft. outside of the outer front vault or area wall of the building. An arched or other proper opening in the wall must be provided for the drain to prevent damage by settlement.

73. If possible, the house drain must be above the cellar floor. The house drain must be supported at intervals of 10 ft. by 8-inch brick piers or suspended from the floor-beams, or be otherwise properly supported by heavy iron-pipe hangers at intervals of not more than 10 ft. The use of pipe-hooks for supporting drains is prohibited.

74. No steam-exhaust, boiler blow-off, or drip-pipe shall be connected with the house drain or sewer. Such pipes must first discharge into a proper condensing tank, and from this a proper outlet to the house sewer outside of the building must be provided. In low-pressure steam systems the condensing tank may be omitted, but the waste connection must be otherwise as above required.

75. The house and drain sewer must be run as direct as possible, with a fall of at least $\frac{1}{4}$ in. per foot, all changes in direction made with proper fittings, and all connections made with Y branches and one eighth and one sixteenth bends.

Size of House Sewer.—76. The house sewer and house drain must be at least 4 ins. in diameter where water-closets discharge into them. Where rain-water discharges into them, the house sewer and house drain up to the leader connections must be in accordance with the following table:

Diameter.	Fall ¼ Inch per Foot.	Fall ½ Inch per Foot.
6 ins.	5,000 sq. ft.	7,500 sq. ft. of drainage of area
7 "	6,900 "	10,300 " " " " "
8 "	9,100 "	13,600 " " " " "
9 "	11,600 "	17,400 " " " " "

77. Full size Y and T branch fittings for hand-hole clean-outs must be provided where required on house drain and its branches.

78. An iron running trap must be placed on the house drain near the wall of the house, and on the sewer side of all connections, except a drip-pipe where one is used. If placed outside the house or below the cellar floor, it must be made accessible in a brick manhole, the walls of which must be 8 ins. thick, with an iron or flagstone cover. When outside the house it must never be less than 3 ft. below the surface of the ground.

79. *A fresh-air inlet* must be connected with the house drain just inside of the house trap; where under ground it will be of extra heavy cast iron. Where possible it will extend to the external air, and finish with an automatic device approved by the Department of Buildings, at a point just outside the front wall of building. The fresh-air inlet must be of the same size as the drain up to 4 ins. For 5- and 6-in. drains it must be not less than 4 ins. in diameter. For 7- and 8-in. drains not less than 6 ins. in diameter or its equivalent, and for large drains not less than 8 ins. in diameter or its equivalent.

[*Note.*—The fresh-air inlet and running trap prescribed by Sections 78 and 79 are not required in many cities, and it is a disputed question whether or not they are desirable.]

Soil-, Waste-, and Vent-pipes.

80. All main, soil, waste, or vent pipes must be of iron, steel, or brass.

89. *The diameters of soil- and waste-pipes* must not be less than those given in the following table:

Main soil-pipes	4	inches
Main soil-pipes for water-closets on five or more floors	5	"
Branch soil-pipes	4	"
Main waste-pipes	2	"
Main waste-pipes for kitchen sinks on five or more floors	3	"
Branch waste-pipes for laundry tubs	$1\frac{1}{2}$	"
When set in ranges of three or more	2	"
Branch waste for kitchen sinks	2	"
Branch waste for urinals	2	"
Branch waste for other fixtures	$1\frac{1}{2}$	"

96. *The sizes of vent-pipes* throughout must not be less than the following:

For main vents and long branches, 2 ins. in diameter; for water-closets on three or more floors, 3 ins. in diameter; for other fixtures on less than seven floors, 2 ins. in diameter; 3-in. vent-pipe will be permitted for less than nine stories; for more than eight and less than sixteen stories, 4 ins. in diameter; for more than fifteen and less than twenty-two stories, 5 ins. in diameter; for more than twenty-one stories 6 ins. in diameter; branch vents for traps larger than 2 ins., 2 ins. in diameter; branch vents for traps 2 ins. or less, 1½ ins. in diameter.

For fixtures other than water-closets and slop-sinks and for more than eight stories, vent-pipes may be 1 in. smaller than above stated.

Traps.

98. Every fixture must be separately trapped by a water-sealing trap placed as close to the fixture outlet as possible.

99. A set of wash-trays may connect with a single trap, or into the trap of an adjoining sink, provided both sink and tub waste outlets are on the same side of the waste line and the sink is nearest the line. When so connected the waste-pipe from the wash-trays must be branched in below the water seal.

100. The discharge from any fixture must not pass through more than one trap before reaching the house drain.

106. All earthenware traps must have heavy brass floor plates soldered to the lead bends and bolted to the trap flange and the joint made gas-tight with red or white lead. The use of rubber washers for floor connections is prohibited.

107. No trap shall be placed at the foot of main soil- and waste-pipe lines.

108. The sizes for traps must not be less than those given in the following table:

Traps for water-closets.	4 inches in diameter
Traps for slop-sinks.	2 " " "
Traps for kitchen sinks.	2 " " "
Traps for wash-trays.	2 " " "
Traps for urinals.	2 " " "
Traps for other fixtures.	1½ " " "

Traps for leaders, areas, floor and other drains must be at least 3 ins. in diameter.

Water-closets.

118. In tenement-houses, lodging-houses, factories, workshops, and all public buildings the entire water-closet apartment and side walls to a height of 16 ins. from the floor, except at the door, must be made waterproof with asphalt, cement, tile, metal, or other water-proof material as approved by the Board of Buildings.

121. The general water-closet accommodations for a tenement- or lodging-house cannot be placed in the cellar.

130. In all sewer-connected occupied buildings there must be at least one water-closet, and there must be additional closets so that there will never be more than fifteen persons per closet.

131. In tenement-houses and lodging-houses there must be one water-closet on each floor, and when there is more than one family on a floor, there will be one additional water-closet for every two additional families.

132. In lodging-houses where there are more than fifteen persons on any floor, there must be an additional water-closet on that floor for every fifteen additional persons or fraction thereof.

133. Water-closets and urinals must never be connected directly with or flushed from the water-supply pipes.

136. Iron water-closets and urinal cisterns and automatic water-closets and urinal cisterns are prohibited.

137. The copper lining of water-closets and urinal cisterns must not be lighter than 10 oz. copper.

138. Water-closet flush-pipes must not be less than $1\frac{1}{4}$ ins. and urinal flush-pipes 1 in. in diameter, and if of lead must not weigh less than $2\frac{1}{2}$ lbs. and 2 lbs. per lineal foot. Flush-couplings must be of full size of the pipe.

Sinks and Wash-tubs.

143. In tenement-houses and lodging-houses sinks must be entirely open, on iron legs or brackets, without any enclosing woodwork.

144. Wooden wash-tubs are prohibited. Cement or artificial stone tubs will not be permitted unless approved by the Board of Buildings.

Testing the Plumbing System.

155. The entire plumbing and draining system within the building must be tested by the plumber, in the presence of a plumbing inspector, under a water or air test, as directed. All pipes must remain uncovered in every part until they have successfully passed the test. The plumber must securely close all openings as directed by the Inspector of Plumbing. The use of wooden plugs for this purpose is prohibited.

156. The water test will be applied by closing the lower end of the main house drain and filling the pipes to the highest opening above the roof with water. The water test shall include at one time the house drain and branches, all vertical and horizontal soil, waste and vent and leader lines and all branches therefrom to point above the surface of the finished floor and beyond the finished face of walls and partitions. Deviation from the above rule will not be permitted, unless upon written application to and approval by the Commissioner of Buildings. If the drain or any part of the system is to be tested separately, there must be a head of water at least 6 ft. above all parts of the work so tested, and special provision must be made for including all joints and connections in at least one test.

157. The air test will be applied with a force-pump and mercury columns under ten pounds pressure, equal to 20 ins. of mercury. The use of spring gauges is prohibited.

158. After the completion of the work, when the water has been turned on and the traps filled, the plumber must make a peppermint or smoke test in the presence of a plumbing inspector and as directed by him.

159. The material and labor for the tests must be furnished by the plumber. Where the peppermint test is used, 2 ozs. of oil of peppermint must be provided for each line up to five stories and basement in height, and for each additional five stories or fraction thereof one additional ounce of peppermint must be provided for each line.

Traps.

A trap is a device which permits the free passage of liquids through it, and also of any solid matters that may be carried by the liquid, while at the same time preventing the passage of air or gas in either direction. Traps used for plumbing

purposes are shaped so that an amount of water sufficient to close the passage and prevent the passage of air will stand in them at all times. The principle of the common trap is shown by Fig. A. The pipe *T* receives the waste from a sink or wash-basin, while the lower end *B* connects with the sewer. Sewer-gas rises in pipe *B*, but is prevented from passing to the fixture by the water which stands in the trap. The depth of water through which gas must pass to effect a passage is termed the "water-seal." The water-seal in the trap, Fig. A, is the distance *S*.

All plumbing pipes which connect with a sewerage system require to be trapped to prevent sewer-gas from passing through

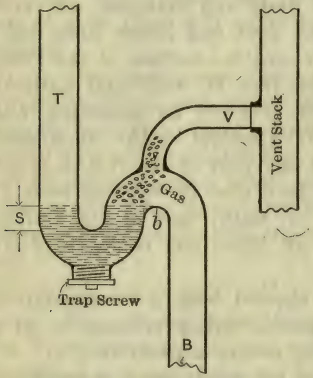


Fig. A

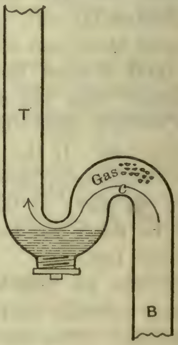


Fig. B

them to the fixture and into the room in which the fixture is located.

Ventilation of Traps.—When a considerable body of water rushes down through a pipe it forms a suction, and if the pipe is made air-tight, this suction is often sufficient to prevent enough water remaining in the trap to form a seal, thus leaving an opening for the passage of sewer-gas as in Fig. B. By connecting the upper bend of a trap with the outside air by means of a pipe, as at *V*, Fig. A, the suction will be stopped, and the water in the pipe *T* will not fall below the level of the outlet at *b*.

Several non-siphoning traps have been patented for the purpose of obviating the necessity of back venting, but they are used to a comparatively limited extent.

There are also several varieties of back-pressure traps, de-

signed to prevent the sewage from flowing back into the house drain. These are in the nature of check-valves, and

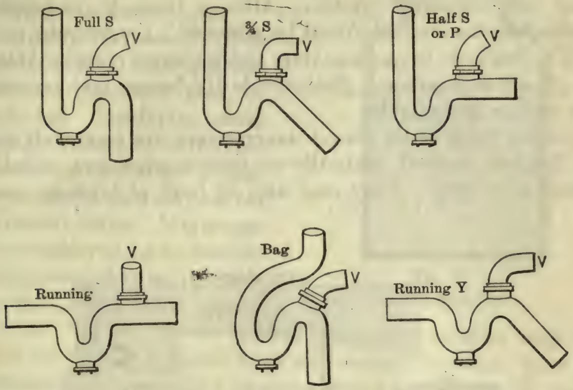


Fig. C
Different Shapes of Traps.

are used principally in seaport towns where tide-water might possibly force the sewage back.

The more common shapes of lead traps used in plumbing, with their trade names, are shown in Figure C. The same shapes are also made in cast iron. The pipes marked *V* are the vent connections. The drum trap shown by Fig. D has a deeper seal than those shown in Fig. C, and is commonly used under kitchen sinks, bath-tubs, and wash-trays. Drum traps are not easily siphoned, even when not vented. The traps for water-closets are commonly formed in the fixture.

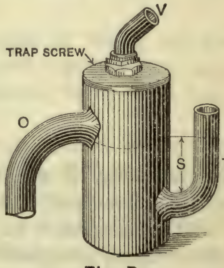


Fig. D
Round Trap.

Grease Traps.—The waste water from kitchen sinks always contains considerable grease, which if permitted to enter the soil-pipe system is liable to clog the pipes by adhering to the walls. In certain localities grease gives much more trouble than in others, due to the chemical composition of the water.

In Colorado and many other places it is necessary to connect the waste from kitchen sinks with a large grease trap, which collects and holds the grease, but permits the water to pass into the sewer system. After a time the accumulated grease fills the trap and must be removed. On account of this it is desirable to use a large trap, and whenever possible it should be placed underground, just outside the house, and as near to the sink as practicable.

Grease traps to be placed underground are commonly made of 24-inch vitrified drain-tile or cement pipe, and should be about 4 ft. deep. They may also be built of brick in cement

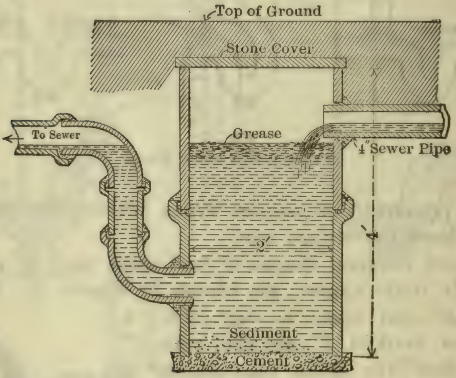


Fig. E

Outdoor Grease Trap.

mortar. Fig. E shows a section through such a grease trap and the inlet and outlet pipes.

When the sink is in a basement or an upper story, or when the building occupies the entire lot, the grease trap must be placed under the sink. When so placed, a round lead trap 12 or 14 ins. in diameter may be used, with a large trap screw in the top for removing the grease. Fig. F shows a section through such a trap and the way in which the connections should be made. A better form of grease trap is made of cast iron. Some city ordinances require that inside grease traps

shall have a chilling jacket for the purpose of more perfectly separating the grease and thus preventing any of it from entering the waste-pipes.

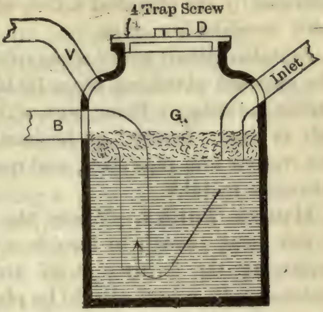


Fig. F
Lead Grease Trap.

Supply-pipes.—These may be of lead, brass, galvanized iron, tin-lined lead, or block tin. Lead pipe offers the least resistance to the flow of water, is easily bent to suit any situation, and easy curves are readily made. It is generally considered more durable underground than galvanized-iron pipe. The grade known as *A*, or "strong," is the lightest that should ever be used, and when the supply is taken from city mains, in which there is a considerable pressure, *AA*, or extra strong pipe, should be used.

Galvanized-iron pipe is probably more extensively used than any other material for water-supply pipes in buildings, except where nickel-plated pipe is required, in which case brass piping is commonly used. Brass pipe used for water-supply should be what is known as iron-pipe size.

Brass piping is preferable to galvanized iron or lead for conveying hot water, and is largely used in the better class of buildings.

Tin-lined iron and lead pipes and pipes of block tin are usually considered as offering the greatest resistance to corrosion or chemical action, and should always be used for conveying ale, beer, and other liquors.

Tin-lined iron pipe is made by pouring melted tin into a wrought-iron pipe. While in a fluid state the tin is inseparably united to the iron, and the result is one solid pipe composed of two metals which *can not be torn apart.* It is essentially different from iron pipe merely dipped in tin, and immeasurably superior to iron pipe lined with a separate tin pipe that will become detached. Its fittings are lined with tin to match. Hot water will not injure it, rats will not gnaw it, and thieves will not cut it out. Either hot or cold water may stand in block-tin pipes and yet be drawn from them pure and free from poison or rust.

Lead-lined pipe is made in the same way and insures deliver-

ing the water to the house just as it comes from the mains unchanged by the chemical action which often results from contact with wrought-iron pipe.

Seamless-drawn nickel-silver tubing is used to some extent for the exposed plumbing pipes in high class residences, office and public buildings. Being pure white metal throughout it can not rub or wear "brassy" or become discolored. It is made in all the regular iron pipe sizes, and necessary fittings are supplied of the same metal.*

House Tanks.—Where the pressure in the street mains is not great enough to furnish a sufficient volume of water for supplying the fixtures at all times, or in cases of a private water-supply, a tank should be placed in the attic, or elevated at least 6 ft. above the highest fixture to be supplied. In some cases the fixtures in the lower story are supplied direct from the street mains, while those in the upper story are supplied from a tank. The advantage of a tank is that it will fill gradually from a very small stream, and thus form a reservoir from which a larger volume can be drawn in a shorter space of time than could be obtained direct from the service pipes.

Storage-tanks should always be provided with an overflow pipe of ample size and when supplied from the street mains the supply should be controlled by a ball cock and float.

Storage-tanks of moderate size are preferably made of wood lined with planished or tinned copper.

Sheet lead, zinc, or galvanized iron should not be used for lining tanks containing water for drinking or cooking purposes, and are not as durable as copper, even when the effect on the water need not be considered.

The size of tank required will depend largely upon the character of the supply. Tanks supplied from the street main in which the pressure is fairly constant need not have a capacity exceeding 160 gallons. Where the water is pumped into the tank by a windmill or hot-air engine, the tank should have a capacity sufficient for a three or four days' supply at least.

Amount of Water Required for Various Purposes.—The amount of water required for household purposes has been found to be about 25 gallons for each person, large or small.

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* For further information consult the Benedict & Burnham Mfg. Co., Waterbury, Conn.

A horse will drink about 7 gallons per day and a cow 5 to 6 gallons per day.

A carriage requires from 9 to 16 gallons for washing.

Size of Supply-pipes.—The proper diameter of supply-pipes depends upon several considerations, such as the number and size of faucets that are likely to be discharging water at the same time, the urgency of the demand, the length of the pipes and number of angles, and upon the pressure.

There is no objection to having a pipe larger than is really necessary, except from the standpoint of cost. Service-pipes should always be one size larger than the tap in the street main.

The following table affords a fair guide for proportioning the supply branches to plumbing fixtures. If the pressure is less than 20 lbs. per square inch the system may be rated as *low pressure*, and if above 20 lbs. as *high pressure*.

Supply Branches.	Low Pressure.	High Pressure.
	Inch.	Inch.
To Bath-cocks	$\frac{3}{4}$ to 1	$\frac{1}{2}$ to $\frac{5}{8}$
Basin-cocks	$\frac{1}{2}$	$\frac{1}{2}$
W. C. flush-tank	$\frac{1}{2}$	$\frac{1}{2}$
W. C. flush-valve	1 to $1\frac{1}{4}$	$\frac{3}{4}$ to 1
Sitz or foot-bath	$\frac{1}{2}$ to $\frac{3}{4}$	$\frac{1}{2}$
Kitchen sinks	$\frac{5}{8}$ to $\frac{3}{4}$	$\frac{1}{2}$ to $\frac{5}{8}$
Pantry sinks	$\frac{1}{2}$	$\frac{1}{2}$
Slop-sinks	$\frac{5}{8}$ to $\frac{3}{4}$	$\frac{1}{2}$ to $\frac{5}{8}$
Urinals	$\frac{5}{8}$ to $\frac{3}{4}$	$\frac{1}{2}$ to $\frac{5}{8}$

With high-pressure systems, dwellings of five or six rooms are sometimes, for economy, supplied entirely through $\frac{1}{2}$-inch pipe.

Minimum Diameter of Waste-pipes.—The following are considered as the smallest diameters allowable for waste-pipes. The diameters required in New York City are given on p. 1264.

Bath and sink wastes, $1\frac{1}{2}$ ins.

Basin and urinal wastes, $1\frac{1}{4}$ ins.

Wash trays, $1\frac{1}{2}$ ins. from each compartment, entered into 4-inch round trap and 2-inch outlet from trap.

Water-closet trap, $3\frac{1}{2}$ ins.

APPROXIMATE SPACING FOR TACKS ON LEAD PIPES.

Size of Pipe, Inches.	Vertical Pipe. Distance Apart, Inches.		Horizontal Pipe. Distance Apart, Inches.	
	Hot.	Cold.	Hot.	Cold.
$\frac{1}{2}$	19	25	14	17
$\frac{5}{8}$	20	26	15	18
$\frac{3}{4}$	21	27	16	19
1	22	28	17	20
$1\frac{1}{4}$	23	29	18	21
$1\frac{1}{2}$	24	30	18	22

Designation of Lead Pipe.—The different thicknesses of lead pipe were formerly designated by letters as in Table B, but are now more commonly designated as in Table A, following, which may be considered as generally accepted by dealers.

TABLE A.—WEIGHTS AND SIZES OF LEAD PIPE.

Calibre.	Weight per Foot. Lbs.	Ozs.	Calibre.	Weight per Foot. Lbs.	Ozs.
$\frac{1}{4}$-in. Tubing.	...	6	$\frac{5}{8}$-in. Ex. ex. Strong	3	8
Fish Seine.	...	15	$\frac{3}{4}$-in. Aqueduct.	1	
$\frac{3}{8}$-in. Aqueduct.	...	8	Ex. Light.	1	8
Ex. Light.	...	10	Light.	2	
Light.	...	12	Medium.	2	4
Medium.	1		Strong.	3	
Strong.	1	8	Ex. Strong. . . .	3	8
Ex. Strong. . . .	2		Ex. ex. Strong.	4	
$\frac{1}{2}$-in. Aqueduct.	...	10	$\frac{7}{8}$-in. Aqueduct.	1	8
Ex. Light.	...	12	Ex. Light.	2	
Light.	1		Light.	2	8
Medium.	1	4	1-in. Aqueduct.	1	8
Strong.	1	12	Ex. Light.	2	
AA.	2		Light.	2	8
Ex. Strong. . . .	2	8	Medium.	3	4
Ex. ex. Strong.	3		Strong.	4	
$\frac{5}{8}$-in. Aqueduct.	...	12	Ex. Strong. . . .	4	12
Ex. Light.	1	4	Ex. ex. Strong.	5	8
Light.	1	12	$1\frac{1}{4}$-in. Aqueduct.	2	
Medium.	2		Ex. Light.	2	8
Strong.	2	8	Light.	3	
Ex. Strong. . . .	3		Medium.	3	12

Calibre.	Weight per Foot. Lbs.	Weight per Foot. Ozs.	Calibre.	Weight per Foot. Lbs.	Weight per Foot. Ozs.
$1\frac{1}{4}$-in. Strong.	4	12	$2\frac{1}{2}$-in. Waste.	4	
Ex. Strong. .	6		Light.	6	
Ex.ex.Strong	6	12	Medium, $\frac{3}{16}$ thick	8	
$1\frac{1}{2}$-in. Aqueduct. . .	3		Strong, $\frac{1}{4}$ "	11	
Ex. Light. . .	3	8	Ex. Strong, $\frac{5}{16}$ "	14	
Light.	4		Ex. ex. Strong, $\frac{3}{8}$ "	17	
Medium.	5		3-in. Waste.	3	8
Strong.	6		Light.	6	3
Ex. Strong. .	7	8	Medium, $\frac{3}{16}$ thick	9	
Ex.ex.Strong	9		Strong, $\frac{1}{4}$ "	12	
$1\frac{3}{4}$-in. Ex. Light. . .	3	12	Ex. Strong, $\frac{5}{16}$ "	16	
Light.	4	8	Ex. ex. Strong, $\frac{3}{8}$ "	20	
Medium.	5	8	$3\frac{1}{2}$-in. Waste.	5	
Strong.	6	8	Strong, $\frac{1}{4}$ thick	15	
Ex. Strong. .	8		Ex. Strong, $\frac{5}{16}$ "	18	
2-in. Waste.	3		4-in. Waste.	5	
Ex. Light. . .	4		Medium.	10	
Light.	5		Strong, $\frac{1}{4}$ thick	16	
Medium.	7		Ex. Strong, $\frac{5}{16}$ "	22	
Strong.	8		Ex. ex. Strong, $\frac{3}{8}$ "	25	
Ex. Strong. .	9		5-in. Waste.	8	
Ex.ex.Strong	10	8			

Coils of supply-pipe weigh about 200 lbs.; Aqueduct about 90 lbs.; Suction-pipe, 100 to 180 lbs. each.

Block-tin pipe is stronger for a given weight per foot than lead- or tin-lined lead pipe. As compared with lead pipe its strength is as $3\frac{1}{2}$ to 1.

Tin-lined and lead-lined iron pipe is made with inside diameters of $\frac{1}{2}$, $\frac{3}{4}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}$, and 2 ins., and in 10-ft. lengths, threaded without couplings. Tin- and lead-lined fittings are also made (see p. 1273).

WEIGHTS AND SIZES OF SHEET LEAD.

Thickness, inches.	$\frac{1}{24}$	$\frac{1}{20}$	$\frac{1}{18}$	$\frac{1}{16}$	$\frac{1}{14}$	$\frac{1}{12}$	$\frac{1}{10}$	$\frac{1}{9}$	$\frac{1}{7}$	$\frac{9}{64}$	$\frac{1}{6}$	$\frac{3}{16}$	$\frac{1}{5}$
Pounds, per sq. ft.	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5	6	7	8	9	10	11	12

TABLE B.—THICKNESS AND STRENGTH OF LEAD PIPES.

Calibre.	Mark.	Weight per Foot.	Thickness.	Mean Bursting Pressure.	Safe Working Pressure.	Calibre.	Mark.	Weight per Foot.	Thickness.	Mean Bursting Pressure.	Safe Working Pressure.
Ins.		lb. oz.	ins.	lbs.	lbs.	ins.		lb. oz.	ins.	lbs.	lbs.
3/8	AAA	1 12	0.18	1968	492	1	A	4 0	0.21	857	214
3/8	AA	1 5	0.15	1627	406	1	B	3 4	0.17	745	186
3/8	A	1 2	0.13	1381	347	1	C	2 8	0.14	562	140
3/8	B	1 0	0.125	1342	335	1	D	2 4	0.125	518	129
3/8	C	0 14	0.11	1187	296	1	E	2 0	0.10	475	118
3/8	......	0 10	0.087	1085	271	1	......	1 8	0.09	325	81
7/16	......	0 9½	0.08	775	193	1¼	AAA	6 12	0.275	962	240
1/2	AAA	3 0	0.25	1787	446	1¼	AA	5 12	0.25	823	205
1/2	......	2 8	0.225	1655	413	1¼	A	4 11	0.21	685	171
1/2	AA	2 0	0.18	1393	343	1¼	B	3 11	0.17	546	136
1/2	A	1 10	0.16	1285	321	1¼	C	3 0	0.135	420	105
1/2	B	1 3	0.125	980	245	1¼	D	2 8	0.125	350	87
1/2	C	1 0	0.10	782	195	1¼	......	2 0	0.095	322	80
1/2	D	0 9	0.065	468	117	1½	AAA	8 0	0.29	742	185
1/2	......	0 10	0.07	556	139	1½	AA	7 0	0.25	700	175
1/2	......	0 12	0.09	625	156	1½	A	6 4	0.22	628	157
5/8	AAA	3 8	0.23	1548	387	1½	B	5 0	0.18	506	126
5/8	AA	2 12	0.21	1380	345	½	C	4 4	0.15	430	107
5/8	A	2 8	0.18	1152	288	½	D	3 8	0.14	315	78
5/8	B	2 0	0.16	987	246	1½	......	3 0	0.12	245	61
5/8	C	1 7	0.117	795	198	1¾	B	5 0	.....	.....	116
5/8	D	1 4	0.10	708	177	1¾	C	4 0	.....	.....	93
3/4	AAA	4 14	0.29	1462	365	1¾	D	3 10	0.125	318	79
3/4	AA	3 8	0.225	1225	306	2	AAA	10 11	0.30	611	152
3/4	A	3 0	0.19	1072	268	2	AA	8 14	0.25	511	127
3/4	B	2 3	0.15	865	216	2	A	7 0	0.21	405	101
3/4	C	1 12	0.125	782	195	2	B	6 0	0.19	360	90
3/4	D	1 3	0.09	505	126	2	C	5 0	0.16	260	65
1	AAA	6 0	0.30	1230	307	2	D	4 0	0.09	200	50
1	AA	4 8	0.23	910	227						

WEIGHT AND SIZES OF PURE BLOCK-TIN PIPE.

Size Inside Diameter in Inches.	Weight per Foot, Ounces.	Size Inside Diameter in Inches.	Weight per Foot, Pounds.
3/16	4	3/4	9, 12, 16
1/4	4, 5, 6	1	12, 16
5/16	4, 5, 6, 8	1¼	20, 28
3/8	4, 5, 6, 8	1½	24 and upwards
1/2	5, 6, 8, 10	2	32 and upwards
5/8	9, 12, 16		

Sewer-pipe.

There are three kinds of sewer- or drain-pipe offered in the market, viz., "Salt Glazed Vitrified Clay-pipe," "Slip Glazed Clay-pipe," and "Cement Pipe." The name of the latter sufficiently indicates what it is without any description.

The "Slip Glazed Clay-pipe" is made of what is known as "fire" (such as fire-brick) clay, which retains its porosity when subjected to the most intense heat. It is glazed with another kind of clay, known as "slip," which, when subjected to heat, melts, creating a very thin glazing, which, being *a foreign substance to the body of the pipe*, is liable to wear or scale off.

"Salt Glazed Clay-pipe" is made of a clay, which, when subjected to an intense heat, becomes vitreous or glass-like; and is glazed by the vapors of salt, the salt being thrown in the fire, thereby creating a vapor which unites chemically with the clay, and forms a glazing, which will not scale or wear off, and is impervious to the action of acids, gases, steam, or any other known substance. It unites with the clay in such a manner as to form *part of the body of the pipe*, and is therefore indestructible.

Salt-glazed pipe can only be made from clay that will vitrify, that is, when subjected to an intense heat will come to a hard, compact body, not porous. And it should be borne in mind that "slip glazing" is only resorted to when the clays are of such a nature that they will not vitrify.

The material of drain-pipes should be a hard, vitreous substance; not porous, since this would lead to the absorption of the impure contents of the drain, would have less actual strength to resist pressure, would be more affected by the frost, or by the formation of crystals in connection with certain chemical combinations, or would be more susceptible to the chemical action of the constituents of the sewerage.

Sewer-pipes should be salt glazed, as this requires them to be subjected to a much more intense heat than is needed for "slip" glazing, and thus secures a harder material.

Cement pipes made without metal reinforcement have not proven sufficiently strong and durable to be used with confidence in any important work. When reinforced with metal, however, they have ample strength, and reinforced cement sewer-pipes of large diameter are used to a considerable extent in Europe.

CARRYING CAPACITY OF SEWER-PIPE.
(Gallons per minute.)

Size of Pipe.	Fall per 100 Feet.							
	1 Inch.	2 Inch.	3 Inch.	6 Inch.	9 Inch.	1 Foot.	2 Feet.	3 Feet.
Inch.								
3	13	19	23	32	40	46	64	79
4	27	38	47	66	81	93	131	163
6	75	105	129	183	224	258	364	450
8	153	216	265	375	460	527	750	923
9	205	290	355	503	617	712	1,006	1,240
10	267	378	463	755	803	926	1,310	1,613
12	422	596	730	1,033	1,273	1,468	2,076	2,554
15	740	1,021	1,282	1,818	2,224	2,464	3,617	4,467
18	1,168	1,651	2,022	2,860	3,508	4,045	5,704	7,047
24	2,396	3,387	4,155	5,874	7,202	8,303	11,744	14,466
27	4,407	6,211	7,674	10,883	13,257	15,344	21,771	26,622
30	5,906	8,352	10,223	14,298	17,714	20,204	28,129	35,513
36	9,707	13,769	16,816	23,763	29,284	33,722	47,523	58,406

For determining the diameter of house sewers, the table on p. 1265 will serve as a good guide. Storm sewers should be proportioned to the area drained.

QUANTITIES OF CEMENT, SAND, AND OF CEMENT MORTAR FOR SEWER-PIPE JOINTS.

(Prepared by J. N. Hazlehurst, C.E.)

For each 100 ft. of sewer (with Portland cement, 375 lbs. net per bbl.)

Size of Pipe, Inch.	L'gth, Feet.	Mortar Cubic Yards.	Proportions: 1 Cement to					
			1 Sand.			2 Sand.		
			Cement, Barrels.	Sand, Cubic Yards.	No. Ft. to Bbl. Cem't.	Cement, Barrels.	Sand, Cubic Yards.	No. Ft. to Bbl. Cement.
6	2½	0.003	0.01248	0.00201	803	0.00855	0.00252	1,168
8	2½	0.038	0.15808	0.02546	633	0.10830	0.03192	923
10	2½	0.058	0.24128	0.03886	410	0.16530	0.04872	605
12	2½	0.089	0.37024	0.05963	270	0.25365	0.07476	394
15	2½	0.123	0.51268	0.08241	195	0.35055	0.10332	285
18	2½	0.167	0.69472	0.11189	144	0.47595	0.14018	210
20	2½	0.237	0.98592	0.15879	101	0.67545	0.19908	148
24	2½	0.299	1.24384	0.20033	80	0.85215	0.25116	117
27	3	0.492	2.04672	0.32964	49	1.40220	0.41328	71
30	3	0.548	2.27968	0.36716	44	1.56180	0.46032	64
36	3	0.849	3.53184	0.56883	29	2.41965	0.71316	41

The maximum rainfall, as shown by statistics, is about an inch per hour (except during very heavy storms), equal to 22,633 gallons per hour for each acre, or 377 gallons per minute per acre.

Owing to various obstructions, not more than fifty to seventy-five per cent. of the rainfall will reach the drain within the same hour, and allowance should be made for this fact in determining size of pipe required.

Plumbing Specialties.

The Kenney Flushometer.—This is a gravity valve designed for flushing all water-closets, urinals, and slop-sinks in a building direct from one tank situated in the attic or where most desirable, thus dispensing with the individual overhead tank.

The pipe from the main tank is run down to the different floors either exposed or concealed and branches taken off from there to the flushometer.

The operation of the flushometer is to pull the handle forward, which raises the main valve off its seat, making a direct connection from the flushometer to the tank. After the handle is released the valve closes slowly of its own accord against a high or low pressure.

It is constructed without springs or cup leathers and closes by gravity; is built to stand the hardest of service, and yet so simple in construction and operation that the same valve is used for all requirements, the only difference being whether it is to work on high or low pressure.

The flushometer is extensively used in the better class of buildings in the Eastern States, including the largest office buildings, factories, schools, hospitals, and the better class of residences, also on steamships and yachts.

Filters.—There are few cities in which the public water-supply is not greatly improved in wholesomeness by being filtered, and in many places filtering is absolutely necessary.

The filter should be large enough so that the velocity of the water passing through it will be low and should be so arranged that the flow of water can be reversed and the accumulated impurities washed into a waste-pipe. In the country a filter suitable for rain-water may be built underground, the filtering process being accomplished by beds of sand and gravel. For

city buildings, however, a portable filter located in the basement should be used. An excellent line of filters is made by Wm. B. Scaife & Sons Co., of Pittsburg. These filters have capacities ranging from 150 to 5,000 gallons per hour. The same company also manufactures a line of patent tripoli filters, especially for drinking and cooking purposes, and ranging in cost from $15 to $200.

Those so-called filters which are made to screw onto the nozzle of an ordinary faucet should be considered merely as strainers, and even for that purpose they soon become foul.

Instantaneous Water-heaters are a great convenience for heating water for baths and wash-basins in buildings in which a constant supply of hot water is not provided, and especially in residences where the cooking is done by gas. They are cylindrical in shape, made of nickel-plated copper, and are usually set on a nickel-plated shelf attached to the wall close to the fixture to be supplied. A heater 10½ ins. in diameter and 30 ins. high will heat 20 gallons of water in eight minutes at a cost of 1½ to 2 cents with gas at $1 per 1,000 cu. ft. A large line of these heaters are made by the Humphry Manufacturing and Plating Co., Kalamazoo, Michigan, for both gas and gasolene, although gas is preferable when it can be had.

The cost of heaters varies from $15 to $45 according to size.

An Automatic Water Heater which maintains water at any desired temperature without attention, provided the building has a supply of live steam, is made by James B. Clow & Sons, the supply of steam being automatically regulated by a thermostat. It will be found especially desirable in hospitals, hotels, apartment-houses, and public institutions. The heater is made in four sizes, with capacities of 1,500, 2,500, 4,000, and 6,500 gallons per hour.

The Climax Cellar Drainer * is a simple device for raising water from 6 to 10 ft. without attention or power, except a supply of steam or water. It is used principally for draining cellars, wheel-pits, furnace-pits, etc., when the same are too low to drain into the sewer. For such places a box or barrel is sunk so that all of the water will run into it, and the drainer is set in this receiver and the discharge pipe run to a sink or open drain. The drainer performs its functions by passing water or steam under pressure through the drainer point or jet,

* Manufactured by Jas. B. Clow & Sons.

thus creating a suction which draws the water from the receiver in which it is placed into the discharge-pipe, and both the jet water and cellar water are discharged together. As long as the city water or steam passes through the drainer-pipe, this suction and discharge continues. The supply of water or steam is turned on or off automatically, so that there is no consumption of city water or steam except when the drainer is removing water. This drainer will operate with pressure of 15 lbs. or more, the heavier the pressure the greater the amount of dead water discharged. When the drainage water does not have to be raised more than 10 ft., this is the most economical apparatus that can be used, as the amount of city water consumed is very small.

The Climax Drainer is made in six sizes, costing from $25 to $160.

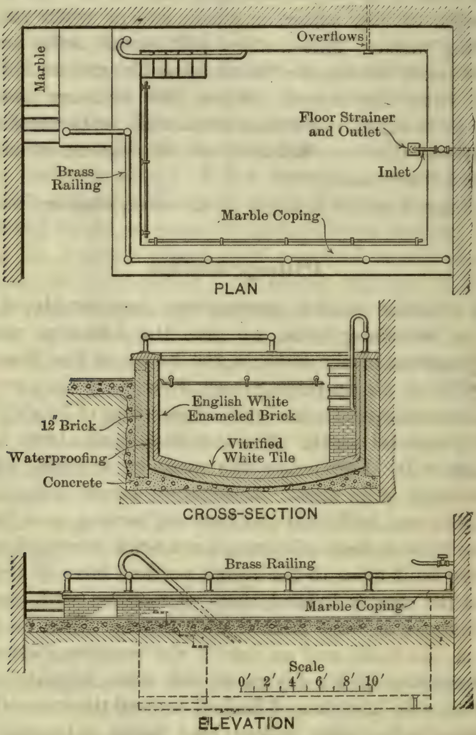
Plunge-baths.

As an example of the construction and details of a small plunge- or swimming-bath, we give the following description and illustrations of the bath in the house of the Racquet and Tennis Club on Forty-third Street, New York City.*

"The swimming-bath has inside dimensions of 15×22 ft. and is about 9 ft. in total depth. It was built in a pit about 19×26 ft. and about 8 ft. deep below the main excavation, which was blasted out of solid rock. A concrete invert a foot or more in thickness was laid over the bottom, serving as a footing on which the 12-inch walls of common red brick were laid in cement. They were built close to the rough vertical faces of the excavation, and the spaces behind them were filled with concrete or cement mortar or were flushed with grout. Then on the inner surface of the walls and on top of the concrete bottom lining a waterproofing of six layers of felt with lapped joints was mopped on with hot tar and flashed around the iron outlet pipe, which also had a wide calked lead flange extending between the layers of felt. On the bottom of this water-proof coat an 8-inch inverted segmental flat floor arch of common brick was laid, and on its skewbacks 4-inch vertical brick walls were built against the water-proofed sides. The bottom was then lined with vitrified white tile and the sides were faced with

* The illustrations and accompanying descriptions are taken by permission from the *Engineering Record* of Nov. 3, 1900.

English white enamelled brick. The tops of the walls were coped with bevelled and moulded white marble slabs which are about 2 ft. above the floor-level and are surmounted at one side and one end by a low heavy rail with twisted ornamental posts, all of brass. A similar horizontal hand-rail is carried



along the inside wall of the bath just above water-level and a curved brass hand-rail is fastened to the wall above the narrow brick and marble stairs at one end. The swimming-bath occupies one corner of the room and its elevated marble platform extends entirely across it, forming a diving platform which is reached by two marble steps.

"All the water-supply is filtered and it can be warmed by injecting steam into the delivery-pipe at the filter. The water

enters through the open upturned end of a 2-inch brass pipe projecting a foot or more through the wall above the top of the bath and delivering a solid jet unless it is reduced by the regulating valve or is formed into a fan-shaped cascade by means of a special nozzle which can be screwed in the open end of the pipe. When the bath is much used a small stream of water is constantly admitted and causes a continual gentle circulation and corresponding overflow, and the entire contents are pumped out and the bath cleaned every two or three days. There are two overflows, an open one about 8 ft. above the bottom and a valved one a foot lower. Mr. C. L. W. Eidlitz was the architect of the house and the waterproofing was done by the T. New Construction Company."

Illuminating-gas.

Varieties of Gas.—Five varieties of gas are now commonly used for lighting and cooking, viz.:

1. *Coal-gas*, which is made by heating bituminous coal in airtight retorts. This is the most common variety of gas furnished for the lighting of cities and towns.

2. *Water-gas*, which is made, usually from anthracite coal and steam, and is quite extensively used in Eastern cities. Gas made by this process contains less carbon than good coal-gas, and consequently does not give as bright a light, although it burns perfectly in heating burners. When used for lighting purposes it is enriched in carbon by vaporizing a quantity of petroleum by heat and injecting it into the hot gas before it leaves the generator.

Pure water-gas is lighter and has less odor than coal-gas.

3. *Natural gas* is obtained from holes or wells which are drilled in the ground. In localities where it can be obtained it furnishes cheap light and fuel. The natural gas obtained in the hard-coal regions develops more heat per cubic foot in burning than any other kind of gas except acetylene. Natural gas is usually under greater pressure in the street mains and house pipes than manufactured gas.

4. *Acetylene-gas.*—Used almost exclusively for the lighting of isolated buildings, or for public buildings in towns or cities where there is no public gas supply, and commonly generated on the premises.

It is formed by bringing water and calcium carbide in contact. Calcium carbide is produced by the electrical fusion of coke and

lime. It is now a commercial article produced in large quantities and sold at a moderate price. It is a very hard substance like dark granite, has a very slight odor, will not burn or explode, and can be handled in any quantity with perfect safety.

The fact that carbide begins to disintegrate and give off acetylene at the slightest touch of moisture makes it practicable to generate the gas in small quantities for single buildings.

Process of Generating Acetylene-gas.—The satisfactory production of acetylene-gas requires a generator which shall feed carbide of sufficient size and weight to be plunged a sufficient depth under the water in the generator-chamber to insure coolness and proper washing. The carbide-chamber must be so arranged and protected that no gas can return to it to be wasted when the chamber is refilled and permeate the house with its smell.

It must feed carbide loosely and in very small quantities, in order to provide for perfect coolness by free access of water to all of the carbide. It must work automatically and with absolute certainty.

Acetylene-gas to be pure must be thoroughly washed. Impure acetylene, as with any other illuminating-gas, means a discoloration of the flame, diminished illuminating power, clogging of pipes and burners with carbon and other foreign matter, and smoky burners, causing blackening of ceilings and tarnished and soiled woodwork and upholstery.

It is now generally agreed that the requirements above outlined can be attained only by a generator of the plunger type.

Portable generators which may be set in the cellar or basement of any building are manufactured in great variety; it is estimated that 100,000 acetylene-gas generators are now in use in the United States. They are made in sizes of 5, 10, 15, 20, and up to 500 lights capacity.

In all machines dropping carbide into water there should be a connection open from the carbide-holding receptacle to the safety-vent run out of doors from the gasometer.

It is claimed that for a given degree of illumination, acetylene is cheaper than "dollar gas." A large residence may be lighted for about $2.50 a month.

To develop the full illuminating power of the gas it is necessary to use a burner-tip having the thinnest slit obtainable, the illuminating power of the gas being about 15 times that of coal-gas, for the same consumption.

The light is a clear white, very nearly resembling sunlight in color and diffusiveness, with none of the red of the incandescent lamp, the orange of the ordinary gas-flame, or the green tone of the incandescent mantle; and it possesses the quality, unique among artificial illuminants, of reproducing even the most delicate shades of color as faithfully as sunlight. Even when used with mantle burners, as it may be with great economy, acetylene light presents a strong dissimilarity from ordinary gas under the same conditions. Acetylene corrodes silver and copper, but does not affect brass, iron, lead, tin, or zinc.

A government specification for a complete apparatus for acetylene gas was published in *Engineering News* of Feb. 4, 1904.

5. *Gasolene-gas* is a mixture of gasolene vapor with air. It is never piped, but is generated close to the burner, and is seldom used for lighting except for street stands, and the like. It is much used for fuel, however.

Gasolene changes from the liquid to the gaseous form under ordinary atmospheric pressure, at temperatures above 40°, the evaporation being very slow at 40°, quite rapid at 70°, and furious at 212°.

If a tank containing liquid gasolene be left open to the air, the liquid will all pass away in the form of gas. If a match be lighted near an open can of gasolene, the escaping gas at once takes fire and communicates the fire to the liquid, causing it to explode with great violence. Although generally considered as dangerous, it is only so when carelessly or ignorantly handled.

To produce 1,000 cu. ft. of gas of good quality requires about 4½ gallons of the best grade of gasolene.

An ordinary burner consumes about 5 cu. ft. per hour.

PIPING A HOUSE FOR GAS.

[Circular issued by the Gilbert & Barker Manufacturing Company.]

Ordinary wrought-iron pipe, such as is used for steam or water, is suitable and proper for all kinds of gas. Galvanized malleable iron fittings, in distinction from plain iron, are very superior. The coating of zinc inside and out effectually and permanently covers all blow-holes, makes the work solid and durable, and avoids the use of perishable cement. Before the pipe is placed in position it should be looked and blown through. It is not infrequent that it is obstructed, and this precaution will save much damage and annoyance. What is known as gas-fitters' cement never should be used. It cracks off easily, in warm

places it will melt, and it can be dissolved by several different kinds of gas. Nothing but solid metals is admissible for confining gas of any kind. When pipes under floors run across floor timbers, the latter should be cut into near their ends, or where supported on partitions, in distinction from being cut in or near the centres of rooms. It is evident that a 10-inch timber notched 2 ins. in the middle is no stronger than an 8-inch. All branch outlet-pipes should be taken from the sides or tops of running lines. Bracket-pipes should run up from below, in distinction from dropping from overhead. Never drop a centre pipe from the bottom of a running line. Always take such outlet from the side of the pipe. The whole system of piping must be free from low places or traps, and decline toward the main rising pipe, which should run up in a partition as near the centre of the building as is practicable. It is obvious that where gas is distributed from the centre of a building, smaller running lines of pipe will be needed than when the main pipe runs up on one end. Hence, timbers will not require as deep cutting, and the flow of gas will be more regular and even. For the same reason in large buildings, more than one riser may be advisable. When a building has different heights of post, it is always better to have an independent rising pipe for each height of post, in distinction from dropping a system of piping from a higher to a lower post, and grading to a low point and establishing drip-pipes. Drip-pipes in a building should always be avoided. The whole system of piping should be so arranged that any condensed gas will flow back through the system and into the service-pipe in the ground. All outlet-pipes should be so securely and rigidly fastened in position that there will be no possibility of their moving when the gas-fixtures are attached. Centre pipes should rest on a solid support fastened to the floor timbers near their tops. The pipe should be securely fastened to the support to prevent lateral movement. The drop-pipe must be perfectly plumb, and pass through a guide fastened near the bottom of the timbers, which will keep them in position despite the assaults of lathers, masons, and others. In the absence of express directions to the contrary, outlets for brackets should generally be 5 ft. and 6 ins. high from the floor, excepting that it is usual to put them 6 ft. in halls and bathrooms. The upright pipes should be plumb, so that the nipples that project through the walls will be level. The nipples should project

not more than $\frac{3}{4}$ in. from the face of the plastering. Laths and plaster together are usually $\frac{3}{4}$ in. thick; hence the nipples should project $1\frac{1}{2}$ ins. from the face of the studding. Drop centre pipes should project $1\frac{1}{2}$ ins. below the furring, or timbers if there be no furring, where it is known that there will be no stucco or centre-pieces used. Where centre-pieces are to be used, or where there is a doubt whether they will be or not, then the drop-pipes should be left about a foot below the furring. All pipes being properly fastened, the drop-pipe can be safely taken out and cut to the right length when gas-fixtures are put on. Gas-pipes should never be placed on the bottoms of floor timbers that are to be lathed and plastered, because they are inaccessible in the contingency of leakage, or when alterations are desired, and gas-fixtures are insecure. The whole system of piping should be proved to be air- and gas-tight under a pressure of air that will raise a column of mercury 6 ins. high in a glass tube. The pipes are either tight or they leak. There is no middle ground. If they are tight the mercury will not fall a particle. A piece of paper should be pasted on the glass tube, even with the mercury, to mark its height while the pressure is on. The system of piping should remain under test for at least a half-hour. It should be the duty of the person in charge of the construction of the building to thoroughly inspect the system of gas-fitting; surely as much so as to inspect any other part of the building. He should know from personal observation that the specifications are complied with. After being satisfied that the mercury does not fall he should cause caps on the outlets to be loosened in different parts of the building, first loosening one to let some air escape, at the same time observing if the mercury falls, then tighten it and repeat the operation at other points. This plan will prove whether the pipes are free from obstruction or not. When he is satisfied that the whole work is properly and perfectly executed, he should give the gas-fitter a certificate to that effect.

The following requirements from specifications published by the Denver Gas and Electric Company are worthy of attention:

Always use fittings in making turns; do not bend pipe. Do not use unions in concealed work; use long screws or right-and-left couplings. Long runs of approximately horizontal

pipe must be firmly supported at short intervals to prevent sagging.

Rules and Table for Proportioning Sizes of House Pipes.*

The table on the opposite page is based on the well-known formula for the flow of gas through pipes. The friction, and therefore the pressure necessary to overcome the friction, increases with the quantity of gas that goes through, and as the aim of the table is to have the loss in pressure not exceed $\frac{1}{10}$ in. water pressure in 30 ft., the size of the pipe increases in going from an extremity toward the meter, as each section has an increasing number of outlets to supply. The quantity of gas the piping may be called on to pass through is stated in terms of $\frac{3}{8}$-in. outlets, instead of cubic feet, outlets being used as a unit instead of burners, because at the time of first inspection the number of burners may not be definitely determined. In designing the table, each $\frac{3}{8}$-in. outlet was assumed as requiring a supply of 10 cu. ft. per hour.

In using the table observe the following rules:

1. No house riser shall be less than $\frac{3}{4}$ in. The house riser is considered to extend from the cellar to the ceiling of the first story. Above the ceiling the pipe must be extended of the same size as the riser, until the first branch line is taken off.

2. No house pipe shall be less than $\frac{3}{8}$ in. An extension to existing piping may be made of $\frac{1}{4}$-in. pipe to supply not more than one outlet, provided said pipe is not over 6 ft. long.

3. No gas-range shall be connected with a smaller pipe than $\frac{3}{4}$ in.

4. In figuring out the size of pipe, always start at the extremities of the system and work *toward* the meter.

5. In using the table, the lengths of pipe to be used in each case are the lengths measured from one branch or point of juncture to another, disregarding elbows or turns. Such lengths will be hereafter spoken of as "sections." No change in size of pipe may be made except at branches or outlets, each "section" therefore being made of but one size of pipe.

6. If any outlet is larger than $\frac{3}{8}$ in. it must be counted as more than one, in accordance with the schedule below:

Size of outlet (inches)........	$\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	2	$2\frac{1}{2}$	3
Value in table	2	4	7	11	16	28	44	64

* The Denver Gas and Electric Company.

TABLE SHOWING THE CORRECT SIZES OF HOUSE PIPES FOR DIFFERENT LENGTHS OF PIPES AND NUMBER OF OUTLETS.

Number of Outlets.	Lengths of Pipes in Feet.								
	⅜-in. Pipe.	½-in. Pipe.	¾-in. Pipe.	1-in. Pipe.	1¼-in. Pipe.	1½-in. Pipe.	2-in. Pipe.	2½-in. Pipe.	3-in. Pipe.
1	20	30	50	70	100	150	200	300	400
2	.....	27	50	70	100	150	200	300	400
3	.....	12	50	70	100	150	200	300	400
4	.....	.....	50	70	100	150	200	300	400
5	.....	.....	33	70	100	150	200	300	400
6	.....	.....	24	70	100	150	200	300	400
8	.....	.....	13	50	100	150	200	300	400
10	.....	.....	.....	35	100	150	200	300	400
13	.....	.....	.....	21	60	150	200	300	400
15	.....	.....	.....	16	45	120	200	300	400
20	.....	.....	.....	.....	27	65	200	300	400
25	.....	.....	.....	.....	17	42	175	300	400
30	.....	.....	.....	.....	12	30	120	300	400
35	.....	.....	.....	.....	.....	22	90	270	400
40	.....	.....	.....	.....	.....	17	70	210	400
45	.....	.....	.....	.....	.....	13	55	165	400
50	.....	.....	.....	.....	.....	.....	45	135	330
65	.....	.....	.....	.....	.....	.....	27	80	200
75	.....	.....	.....	.....	.....	.....	20	60	150
100	.....	.....	.....	.....	.....	.....	.....	33	80
125	.....	.....	.....	.....	.....	.....	.....	22	50
150	.....	.....	.....	.....	.....	.....	.....	15	35
175	.....	.....	.....	.....	.....	.....	.....	.....	28
200	.....	.....	.....	.....	.....	.....	.....	.....	21
225	.....	.....	.....	.....	.....	.....	.....	.....	17
250	.....	.....	.....	.....	.....	.....	.....	.....	14

7. If the exact number of outlets given cannot be found in the table, take the next larger number. For example, if seventeen outlets are required, work with the next larger number in the table, which is 20.

8. If, for the number of outlets given, the exact length of the "section" which feeds these outlets cannot be found in the table, the next larger length corresponding to the outlets given must be taken to determine the size of pipe required. Thus, if there are eight outlets to be fed through 55 ft. of pipe, the length next larger than 55 in the eight-outlet line in the table is 100, and as this is in the $1\frac{1}{4}$-in. column, that size pipe would

be required. Under Rule 7 the same size pipe would be required for seven outlets.

9. For any given number of outlets, do not use a smaller size pipe than the smallest size that contains a figure in the table for that number of outlets. Thus, to feed 15 outlets, no smaller size pipe than 1 in. may be used, no matter how short the "section" may be.

10. In any piping plan, in any continuous run from an extremity to the metre, there may not be used a longer length of any size pipe than found in the table for that size, as 50 ft. for $\frac{3}{4}$ in., 70 ft. for 1 in., etc. If any one "section" would exceed the limit length, it must be made of larger pipe. Thus, 6 outlets could not be fed through 75 ft. of 1-in. pipe, but $1\frac{1}{4}$ in. would have to be used. When two or more successive "sections" work out to the same size of pipe and their total length or sum exceeds the longest length in the table for that size pipe, make the "section" nearest the metre of the next larger size. For example, if we have 5 outlets to be supplied through 45 ft. of pipe, and these 5 and 5 more, making 10 in all, through 30 ft. of pipe, we should find by the table that 10 outlets through 30 ft. would require 1-in. pipe, and that 5 outlets through 45 ft. would also require 1-in. pipe, but as the sum of the two sections, 30 plus 45 equals 75 ft., is longer than the amount of 1 in. that may be used in any continuous run, the 30-ft. section, being the one nearer the metre, must be made of $1\frac{1}{4}$-in. pipe. The application of the limit in length of any one size in a continuous run may also be shown as follows: Eight outlets will allow of 13 ft. of $\frac{3}{4}$-in. pipe in the section between the eighth and ninth outlet (counting from the extremity of the system toward the metre), provided that this 13 ft. added to the total length of $\frac{3}{4}$-in. pipe that may have been used between the extremity of the run and the eighth outlet does not exceed 50 ft., which, according to the table, is the greatest length of $\frac{3}{4}$ in. allowable in any one branch of the system. Therefore, up to the eighth outlet, 37 ft. of $\frac{3}{4}$-in. pipe could have been used, and yet allow 13 ft. of $\frac{3}{4}$ in. to be used in the section between the eighth and ninth outlet. If more than 37 ft. had been used, then the entire 13 ft. between the eighth and ninth outlets would have to be of 1-in. pipe.

11. Never supply gas from a smaller size pipe to a larger one. If we have 25 outlets to be supplied through 200 ft. of pipe, and these 25 and 5 more, making 30 in all, through 100 ft. of pipe we should find by the table that 25 outlets through

200 ft. would require 2½-in. pipe, and 30 outlets through 100 ft. would require 2-in. piping, but as under this condition a 2-in. pipe would be supplying a 2½-in., the 100 ft. section must be made 2½ in.

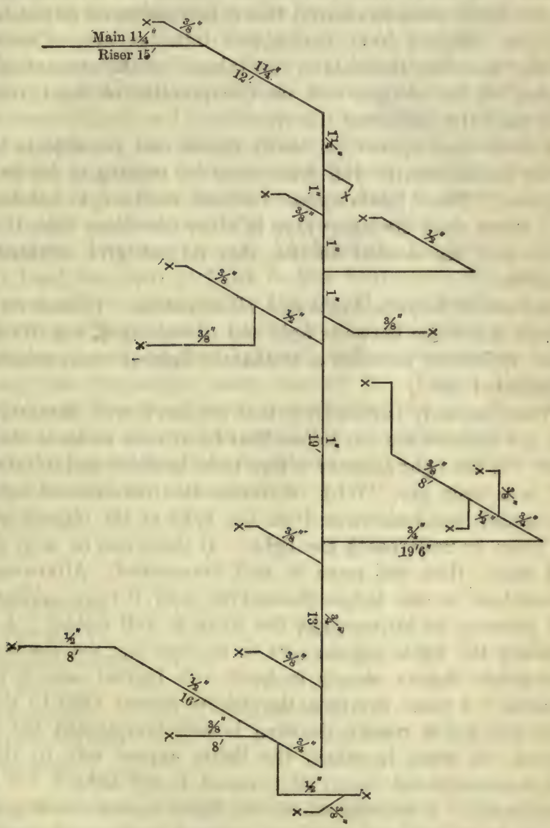


Diagram of Gas-piping.

The sizes of pipes in the above diagram are in accordance with the foregoing rules and table.

Notes on Lighting and Illumination.*

When we say that we *see* an object, we mean that the light which is reflected from its surface strikes our eye. When we see an object twice as clearly, this is because twice as much light is being reflected from that object into our eye. Our seeing objects, therefore, depends on two things: on the amount of light shining on the surface and on the quantity of light reflected back from the surface of the eye.

A room may *appear* brilliantly *lighted* and yet objects looked at not be sufficiently well *illuminated* for reading or for working purposes. These lights *appear* brilliant to the eye, but because they throw their strongest rays in other directions than those in which they are needed for use, they do not give efficient illumination.

Distinction between Light and Illumination.—There is not only a great difference between *light* and *illumination*, but there is a great difference between a brilliantly lighted room and a well illuminated one.

When anybody is asked whether a room is well illuminated or not, the chances are ten to one that he at once looks at the light itself. If the light appears to him to be brilliant and dazzling, he will invariably say, "Why, of course, the room is well lighted." He should first look away from the light at the objects around the room or underneath the light. If these can be seen clearly and easily, then the room is well *illuminated*. Afterwards he should look at the lights themselves, and if they appear soft and pleasing to his eyesight the room is well *lighted*. A room in which the lights appear soft to the eye and yet the eye can distinguish objects clearly is both well lighted and well illuminated. A room in which the objects appear clear to the eye while the lights remain dazzling is well illuminated but badly lighted. A room in which the lights appear soft to the eye and the objects not clearly illuminated, is well lighted, but badly illuminated. A room in which the lights appear dazzling to the eye and the surrounding objects or those underneath not clear to the eyesight is both badly lighted and badly illuminated.

An axiom in good artificial illumination is to keep the illumination of objects around as strong as possible, but the intensity or brilliancy of the lights as low as possible. By doing the first

* Compiled from papers by Wm. Lincoln Smith, Van Renssellaer Lansingh, and a pamphlet published by the Holophane Glass Co.

we enable the eye to see better, by doing the second we enable the eye to feel better, and suffer less from temporary discomfort or permanent injury.

It is not generally understood that a light which is dazzling and brilliant to the eyesight may not be giving as much illumination as another source of light which appears soft, or even dim, by comparison. A bare 50-candle-power incandescent gas-light put next to a 100-candle-power light inside of an opal globe will appear more brilliant and dazzling than the second, but will not be giving as efficient an illumination. The actual illuminating power of the two lights is about equal, but the intensity or dazzling quality of the bare 50-candle-power light is twenty times that of the 100-candle-power light in the opal globe.

Relative Intensity of Artificial Lights. — The present legal standard of light in this country is the British standard candle weighing six to the pound and burning at the rate of 120 grains per hour. Owing to the very wide variations from various causes in the light of candles, measurements of brilliancy are frequently made against the Hofner Alteneck lamp burning acetate of amyl and certified by the Reichanstalt. It may be assumed to equal 0.88 standard candle.

The following table gives the candle-power per square inch for the older and newer illuminants: *

Flat gas-flame	4 to 8	candles per sq. in.
Oil-lamps	3 to 8	" " " "
Candle	2 to 4	" " " "
Welsbach mantle	20 to 25	" " " "
Acetylene-gas	75 to 125	" " " "
Incandescent-lamp	100 to 200	" " " "
Nernst lamp	1,000 to 1,500	" " " "
Crater open arc	25,000 to 50,000	" " " "
Sun on the horizon	2,000†	" " " "
Sun in zenith	190,000	" " " "

From these figures it will be readily seen that much greater care is necessary than formerly in fixing the location of lights, especially when it is remembered that the average eye is liable to be more or less seriously disturbed if subjected for any length of time to the direct rays of a light source of much higher intrinsic brilliancy than an ordinary gas-flame of about five

* Wm. Lincoln Smith, engineer.

† Van Rensselaer Lansingh, engineer.

candles per square inch. From this consideration it at once follows that naked lights of the more modern type should be kept carefully out of range of direct vision, or if necessity requires that they should fall within the range of vision they should be so screened as to reduce the intrinsic brilliancy to within proper limits.

Another measurement of the intensity of illumination is the "candle-foot," which is the illumination given by one candle at a distance of 1 ft.

A candle-foot light is considered a good intensity for reading purposes. *The intensity of light varies inversely as the square of the distance*, or candle-feet = candle-power divided by square of distance in feet.

Thus a 16-candle-power lamp has an intensity of 16 candle-feet at a distance of 1 ft., 4 candle-feet at a distance of 2 ft., and 1 candle-foot at a distance of 4 ft.

Quantity of Light Required.* — The quantity of light to be supplied is usually estimated in candle-power per square feet of area, or per cubic foot of space. Fontaine showed that the latter method is in the majority of instances more nearly correct. In a drawing-room with medium-tinted walls, for instance, 0.015 candle-power per cubic foot is about right and in larger halls it is advisable to figure about 0.02 to 0.03 candle-power per cubic foot. The following table will give some idea of the values to be found in practice:

Apartment.	Volume. Cubic Feet.	Candle-power.	Candle-power per Cubic Foot.
1. Museum.	330,000	8,000	.024
2. Public hall	12,425	1,000	.080
3. Town hall.	48,000	1,376	.028
4. Legislative hall.	143,560	7,560	.052
5. Body opera-house.	324,760	11,400	.034
6. Theatre.	125,550	2,340	.019
7. Colonial church.	80,000	1,600	.020

In this list the second is wastefully bright, the others are all about right.

It must be remembered that all globes and shades absorb a

* Wm. Lincoln Smith, engineer.

certain amount of light and that it is advisable to increase the allowance as figured so as to correct for this loss.

"Assuming the 16-candle-power lamp as the standard, it is generally found that two 16-candle-power lamps per 100 sq. ft. of floor space give good illumination, three very bright, and four brilliant. These general figures will be modified by the height of ceiling, color of walls and ceiling, and other local conditions." *

SPACE ILLUMINATED BY ENCLOSED ARC LAMPS.†

Space to be Illuminated.	Square Yards per 450-Watt Lamp.
Outdoor areas	2000–2500
Train sheds	1400–1600
Foundries (general illumination)	600– 800
Machine shops	200– 250
Thread and cloth mills	200– 230

Means for Reducing Intrinsic Brilliancy—Globes.—In practically all cases of interior illumination it is necessary to reduce the intrinsic brilliancy by the use of some type of diffusing shade or globe, and frequently to further alter the direction of the rays of light by some type of reflector, etc. There are on the market shades and globes which aim to accomplish each of these two results independently of the other, one or two forms which aim to do both at once, and a great number which accomplish neither result to any degree of satisfaction are wasteful of light and many of them cannot even claim to be ornamental.

Opal, opaline, or ground glasses are very good diffusers, but they waste from 30 to 60 per cent. of the light. After diffusing the light they cannot deflect or direct the rays in such directions as they may be needed for use. Their only use, therefore, is in softening the source of light so as to render it less injurious to the eyesight, for they have no power to increase the efficient illumination in any special direction. By a properly calculated and worked-out system of prism glass globes any light may be diffused over a large surface and its intensity softened, while at the same time these diffused or

* H. C. Cushing, Jr., in "Practical Lessons in Electricity."
† International Library of Technology, Vol. 13.

softened rays are deflected into directions where they are needed for use, thereby increasing the efficient illumination.

At present there is only one system of prism glass globes known to the author that carries out these purposes. This is the invention of Messrs. Blondel and Psaroudaki of Paris, who have called it the Holophane system of compound prism glass.

Absolutely transparent glass is used. The inner surface of the glass is given over to carefully calculated flutings or prisms used solely for diffusing or softening the light without loss of power. On the outside surface are prisms calculated for deflecting these diffused rays into directions where needed.

In practice, Holophane glass, made into globes and shades, when placed over a light, will render a dazzling light soft and healthful, while increasing its effective illuminating power.

These globes are made of three classes, or of three different shapes, each shape designed for a separate purpose, as for desk, general, or large interior illumination.

The Meridian Lamp.—The General Electric Company has introduced a new lamp, which they have named the "Meridian Lamp," which is a specially designed incandescent lamp

ILLUMINATING DATA FOR MERIDIAN LAMPS.

Class Service.	Light Intensity in Candle-feet.	No. 1 Lamp (60 Watts).		No. 2 Lamp (120 Watts).	
		Height of Lamp and Diameter of Uniformly Lighted Area.	Distance between Lamps when Two or More are Used.	Height of Lamp and Diameter of Uniformly Lighted Area.	Distance between Lamps when Two or More are Used.
		Feet.	Feet.	Feet.	Feet.
Desk or reading-table	3	2.9	4.9	4	7
	2	3.5	6	5	8.5
	$1\frac{1}{2}$	4	7	5.75	9.8
General lighting	1	5	8.5	7	12
	$\frac{3}{4}$	5.75	9.8	8.2	13.9
	$\frac{1}{2}$	7	12	10	11

with a suitable reflector and ornamental collar detachable from the lamp. The lamp bulb is spherical in shape, and usually frosted. The light from the lamp is of high brilliancy, rendered soft and white by the diffusing action of the sand-

blasted bulb and reflector. The illumination is uniform over an area having a diameter equal to the height from the plane on which the illumination is measured. The lamp is made in two sizes, No. 1, $3\frac{3}{4}''$-bulb, 25 candle-power, consuming 60 watts, and No. 2, $5''$-bulb, 50 candle-power, consuming 120 watts. The prices of the bulbs for renewal are 40 and 60 cents respectively. The lamps should always be suspended, preferably from the ceiling.

The Nernst Lamp.—This is a new form of incandescent lamp which derives its name from the inventor, Dr. Walther Nernst, an eminent German scientist. The distinguishing features of the Nernst lamp are its filament or glower, and the means for making the glower conductive. The glower operates in the open air, its removal and replacement may be readily accomplished, and at ordinary temperatures it is a non-conductor of electricity. A heater, separate therefrom, is therefore provided for giving it an initial temperature sufficient to make it conductive. When it becomes conductive by external heat the current traversing the glower not only causes it to emit light, but also to develop internally sufficient heat to maintain it in a conductive condition, the action of the preliminary heater being discontinued. For a given illumination the Nernst lamp requires only about one half the amount of electrical energy required by ordinary incandescent lamps, and about the same as that of the enclosed arc lamp.

Nernst lamps are made in two types, viz.: the 110-volt type, adjustable for any voltage from 100 to 120, and the 220-volt type, adjustable for any even voltage from 200 to 240. The former type is made in two sizes and the latter type in five sizes.*

Color of Illuminants.†

The question of color plays an important part in the study of illumination. For some purposes, where it is desired to produce the effects of daylight as nearly as possible, the arc lamp, when the violet rays are properly filtered out, plays an important part. In general, however, the color of the arc is cold, and for a room where the effect is to produce a cheerful and warm appearance, an illuminant with more of the red rays, such as the incandescent

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* Additional data may be obtained from the Nernst Lamp Company, Pittsburg, Pa.

† Van Rensselaer Lansingh.

lamp or ordinary gas, is very much to be preferred. The predominant color in a few of the most important illuminants is:

Sun at noon—white.

Sun near sunset—reddish.

Enclosed arc, low voltage—white.

Open arc—bluish white to violet.

Nernst lamp—white.

Acetylene—white.

Incandescent electric—yellowish white.

Mantle burner—white with a tinge of green.

Open-flame gas—orange-white.

Kerosene lamp—orange-white.

Candle—orange-yellow.

The amount of light reflected from surfaces is also largely dependent on the condition and color of such surfaces. It is very necessary in calculations, where the walls or decorations play any part, to know the character of the same; thus white paper reflects from 70 to 80 per cent.; yellow wall-paper gives only about half that amount; emerald-green about 18 per cent.; black paper about 5 per cent., deep-blue paper about 3 per cent., while black velvet gives only about $\frac{4}{10}$ of 1 per cent. As these coefficients of diffuse reflections play an important part in all attempts at calculation, it is necessary, in order to do the best work, to know rather closely the character of the decorations.

The Diffusion of Light through Windows.

Abstracts from report of Mr. Charles L. Norton, on an elaborate series of tests made at the Mass. Institute of Technology.*

The results of the tests on a score or more of different glasses may be stated briefly as follows:

We may increase the light in a room 30 ft. or more deep to form three to fifteen times its present effect by using "Factory Ribbed" glass instead of plane glass in the upper sash. By using prisms we may, under certain conditions, increase the effective light to fifty times its present strength. The gain in effective light on substituting ribbed glass or prisms for plane glass is much greater when the sky-angle is small, as in the case of windows opening upon light shafts or narrow alleys. The increase in the strength of the light directly opposite a window in which

* From Report No. III, Insurance Engineering Experiment Station, Sept., 1902.

ribbed glass or prisms have been substituted for plane glass is at times such as to light a desk or table 50 ft. from the window better than one 20 ft. from the window had previously been lighted.

The kinds of glass tested were as follows:

1. Ground glass of different degrees of fineness.
2. Rough plate or hammered glass.
3. Ribbed or corrugated glass, with five, and eleven and twenty-one ribs to the inch, the corrugations being sinusoidal in outline (as in Fig. A) and the back of the plate smooth.
4. Glass known as "Maze," "Florentine" or "Figured," in which a raised pattern is worked upon one side, practically roughening the whole surface.
5. "Wash-board" glass, corrugated, with twenty-one ribs to the inch on one side and five ribs to the inch on the other side, the ribs being parallel.
6. "Skylight" glass, which has five ribs to the inch on each side, groove on one side being opposite the rib on the other, giving a sinuous section (Fig. B).
7. "Ripple glass," with rippled surfaces on both sides; of very beautiful appearance and a clear white color.
8. Glass ribbed on one side and figured on the other.
9. Ribbed glass with a wire net pressed into it, to increase its resistance to fire.

Of these several specimens, one or two may be dismissed with brief mention. Ground glass is of little value, except as a softening medium for bright sunlight. Its rapidly increasing opaqueness with moisture and dust makes it undesirable as a window glass. The common rough plate has very little action as a diffusing medium, giving no perceptible change in the effective light. "Ripple glass" has great value as a diffusing medium in small rooms with nearly open horizon. Of the ribbed glasses, the fine "Factory Ribbed," with twenty-one ribs to the inch, is distinctly the best, not in all probability because of the fineness, but because of the greater sharpness of the corrugation. The "Ribbed Wire" glass is about twenty per cent. less effective than the ordinary "Factory Ribbed" glass. The addition of a second corrugation upon the back of the plate giving the "Skylight" and "Wash-board" glass is of no apparent value. The raised pattern imprinted upon one surface of the glass, as in the case of the "Maze," gives the *widest diffusion*, especially in bright sunlight. A raised figure, when worked upon the back

of the "Ribbed" glass, renders it less offensive to the eye in bright sunlight, but less effective in deep rooms. The only

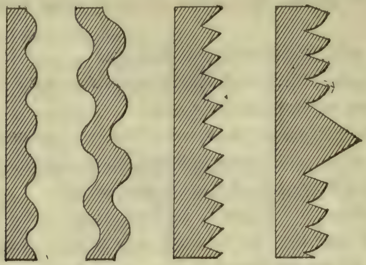

Fig. A Fig. B Fig. C Fig. D

glasses of this group which it is worth while, then, to discuss further are the "Factory Ribbed" and the "Maze" glass.

The second group comprises the following glasses:

1. The Luxfer prisms.
2. The Solar prisms.
3. The Daylight prisms.
4. The glass of prismatic section made by the Mississippi Glass Company.
5. Three-way prisms.
6. Maltby prisms.

The Luxfer prism consists of a plate smooth upon one side and deeply notched upon the other (Fig. C), the teeth or prisms being of very flat, smooth faces and of brilliant appearance. The glass is clear white, and the prisms used in canopies and in the major part of the vertical glazing are made in tiles or plates about 4 ins. square. Tiles are built up in large sheets in frames of copper or brass, so made as to give to the sheets of tiles a strength and durability far in excess of a single sheet of the same size. The Luxfer prisms are now being made for factory use in large sheets, as well as in the small tiles. The Solar prisms are made in small tiles, which are held together in a metal frame to make large sheets. The main difference between the Solar and Luxfer prisms is that the under face of the former prism is curved instead of plane, as in Fig. D. The Daylight prisms tested were made in large sheets and of approximately the same cross-section and general appearance as the Luxfer prisms for factory use. No tiles of Daylight prisms were tested, as none came to hand in time for the test. The Mississippi prism glass is

much like the other prisms in cross-section, but the ridges or prisms do not run across the plate in a straight line, but in a wavy or sinuous line. I cannot detect any advantage arising from this over the straight-edge prism.

Conclusions.—*First.* The conditions in a room less than 15 ft. deep are such that, except with a skylight of less than 45°, it is not advisable to alter the general course of the light by using a prismatic or ribbed glass. A nearly hemispherical diffusion, such as is given by the "Maze" or "Ripple," is ordinarily preferable.

Second. When a room is from 20 ft. to 60 ft. deep, or even more, and has a skylight of 60° or less, the ribbed and prismatic glass gives a very great gain in effective light. The gain in brilliancy is such as to make a basement with prism canopies as light as a second story with plane glass.

Rooms with windows opening upon light-shafts and narrow alleys with very limited sky, where the available light is now small, may have the light 20 ft. back from the window increased

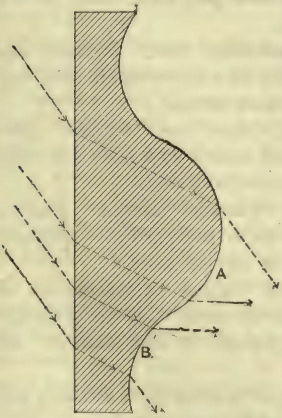

Fig. E

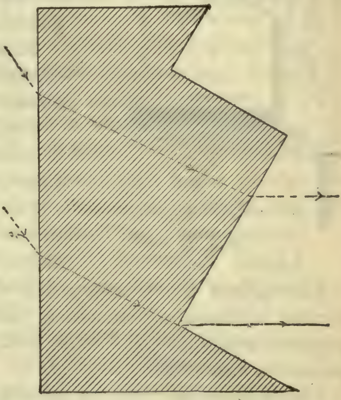

Fig. F

ten or twenty times by using prisms; and, by using canopies of prisms, it is sometimes possible to strengthen the light from fifty to one hundred times.

With sky-angles of 30°, or less, and in deep rooms, the relative efficiency of the prism tile increases greatly.

The refraction of the incident ray in a case of the ribbed glass and prism is shown by Figs. E and F.

"Ribbed" and "Maze" glass are of very great value in softening the light, especially in the case of such windows as are exposed to the direct sun, aside from their effectiveness in strengthening the light at distant points. With the "Maze" glass, the artist may have, in all weather and in all directions, what is in effect a much-desired "north light." The photographer may have in this way as well diffused a light as he now has with cloth screens or shades, with a much greater intensity. To be efficient in rooms 20 ft. deep or more, ribbed glass should be set with its ribs horizontal, and where the sunlight falls upon it, it should be provided with thin white shades. All inferences drawn from the test are made upon the assumption that the windows are to be glazed with diffusing glass only in the upper half, which is the common practice. If the lower sash is to be glazed with diffusing glass as well, a further increase of about twenty-five per cent. may be expected.

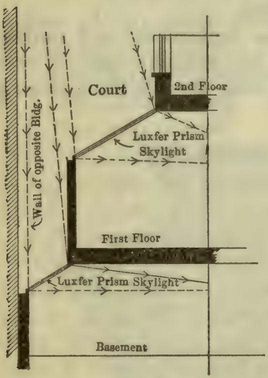

Fig. G

Considering both expense and efficiency, the following general suggestions are given:

Use "Maze" or "Ripple" glass in small rooms or offices not more than 15 or 20 ft. deep.

Use "Factory Ribbed" glass in rooms 30 to 50 ft. deep, with sky-angles of 60° or more.

Use prisms or "Factory Ribbed" glass, in sheets, in all vertical windows in rooms more than 50 to 60 ft. deep, with sky-angle of less than 45°. With a sky-angle of less than 30° use prisms in canopies.

Fig. G shows an effective method of lighting the basement and first story where the light must come from a court.

ELECTRICITY.*

Definitions.—*Electricity* is the name given to that invisible agent which causes all electrical phenomena. Just what this agent is, is unknown. It seems probable that all electrical phenomena are due to a peculiar state or stress of a medium called ether.

Electrical science is founded upon the effects produced by the action of certain forces upon matter.

Electricity may appear either to reside upon the *surface* of bodies as a charge under high pressure or to flow through their substance as a current under comparatively low pressure.

The former is called *static electricity* and the latter *dynamic electricity*.

That branch of electrical science which treats of static electricity is termed *electrostatics* and that which treats of the action of electric currents is termed *electrodynamics*.

Static electricity is produced by friction and is used principally in medicine.

An *electrostatic battery* consists of a number of Leyden jars whose inside coatings are all connected together and whose outside coatings are all connected to the earth.

Voltaic electricity is a term applied to electricity developed by chemical action in a voltaic cell, or battery. Such batteries develop a continuous current of electricity, and hence voltaic electricity is but a sub-branch of dynamic electricity. Electric currents may be obtained by chemical action, heat, or induction.

The Electromagnet.—When an electric current is passed through a coil of wire, the coil becomes equivalent to a magnet and possesses the same properties.

When a core of soft iron is inserted in such a coil it becomes an *electromagnet*. The core is magnetized only when the current is flowing in the coil, and it is to this fact that the practical value of the electromagnet is due. The principle of the electromagnet is employed in the construction of telegraphic instruments, dynamos, electric bells, etc. Electric clocks are also governed by the action of electromagnets.

Dynamos generate current by the rovolving of their arma-

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* In the preparation of this subject the author has had the valued assistance of Mr. Geo. A. Stiles. Electrical Engineer. Denver.

tures in a magnetic field. The armature is an electromagnet with the wire wound parallel to its axis and so connected to a commutator in direct-current machines and to collector rings in alternators that the current may be taken off by brushes applied to the commutator or collecter rings as the case may be.

Types of dynamos may be divided into two divisions, being distinguished by the nature of the current they are to supply—the one type continuous or direct current, the other alternating or rapidly reversing the directions of current.

Flow of Electricity.—Electricity, although commonly described and referred to as *flowing* through a circuit, does not actually *flow*. There is no transfer of matter along the circuit. A wire carrying a current looks the same as one that is not, and that electricity is present is only evident by the heating, chemical, or magnetic effects produced. For practical purposes, however, it is convenient to consider electricity as flowing.

As water flows from a higher to a lower level, so electricity flows from a high *potential* to a lower potential.

Potential is the electrical difference between the plates of a battery or the poles of a dynamo or induction coil; it is analogous to "head," or "pressure," in hydraulics.

Electromotive Force.—As stated above, whenever a difference of electrical potential exists between two points of a circuit it causes a current to flow, and this difference of potential, or the force to which it gives rise, is called electromotive force, commonly designated by the letters E.M.F. or simply E.

The terms potential difference and electromotive force are commonly used with the same meaning.

The unit of electromotive force is the **volt**, and the head, or *pressure*, which produces the current is the *voltage*, high or low voltage meaning that the E.M.F. is measured by a large or small number of volts. In common language the terms pressure, voltage, difference of potential, and electromotive force are synonymous.

The strength of current in a conductor, corresponding to rate of flow, for air or water is the *quantity of electricity* which passes any point in the circuit in a second and is measured in **amperes.** The quantity of electricity conveyed in a given time is the product of the strength of the current by the time it continues.

The quantity of electricity which passes any cross-section of the conductor in one second when the current strength is *one ampere* is called a **coulomb.**

The quantity or amount of electricity in coulombs is equal to the current strength in amperes multiplied by the time in seconds. Thus with a current of 4 amperes flowing for three seconds the quantity delivered is 12 coulombs.

The coulomb is also called the *ampere-second.*

An *ampere-hour* represents an amount of electricity equal to 1 ampere flowing one hour, or 3,600 seconds, and is consequently equal to 3,600 coulombs.

Load.—The term *load* as used in electricity generally refers to the current that is required either for supplying lamps or motors. The load of a motor is the mechanical energy required of it.

Resistance is that property of matter in virtue of which bodies oppose or resist the free flow of electricity and is analogous to friction or obstructions in water-pipes.

The specific resistance of a substance is the resistance of a portion of that substance of unit length and cross-section at a standard temperature, and is an inherent property of the substance or material. The specific resistance of any material must first be determined by experiment.

The resistance of a conductor varies directly as the length, inversely as the cross-sectional area or as the square of the diameter, if the conductor is in the shape of a wire, and depends upon the specific resistance of the material.

Thus the resistance of a wire 100 ft. long is twice as great as another of the same cross-section and material 50 ft. long; but if the sectional area of the first is twice that of the second, then both wires will have the same resistance.

If a circuit is made up of several different materials joined in series with each other, the resistance of the circuit is equal to the sum of the resistances of its several parts.

The unit of resistance is the **ohm,** which is the resistance of a uniform column of mercury 106.3 centimeters long and 14.4521 grams in mass at the temperature of melting ice.

The resistance of a piece of round copper wire .001 in. in diameter and 1 ft. long is 10.8 ohms.

All metals have their resistance increased by a rise of temperature.

Heating Effects of Current.—The passage of electricity through a circuit raises the temperature of the circuit a certain amount.

Joule's law is as follows: "The heating power of a current is proportional to the product of the square of its strength and the resistance of the circuit through which it passes."

The heating of a wire carrying a current is made use of for lighting, for electric heaters, for exploding charges of powder, dynamite, etc. The use of *fuses* for the protection of electric circuits is also based on this principle.

Electric welding is accomplished by passing a powerful current through two bars pressed together. The heating of the junction fuses the metal and the rods become welded.

Energy.—The unit of mechanical energy is the raising of 1 lb. 1 ft. The unit of electrical work is the energy expended by 1 ampere in 1 second in overcoming the resistance of 1 ohm and is called the **joule.** The joule may also be defined as the energy expended when 1 coulomb is carried through a distance between which the difference of potential is 1 volt.

Electrical Power.—The unit of mechanical work is the *foot-pound per minute.* In electrical work the unit is the *joule per second* = **1 watt.** The watt is also sometimes called the *volt-ampere.* One *kilowatt* = 1,000 watts.

The *kilowatt-hour* is a unit of energy and is the energy expended in one hour when the power is 1 kilowatt.

746 watts = 1 electrical horse-power and is equivalent to 1 mechanical horse-power.

Notation of Electrical Units.—The various electrical units are commonly represented by the letters given in the following table, those in parenthesis being sometimes used instead of the letter which precedes:

Volt, the electromotive force } E. or E.M.F.	Watt, the unit of power, W (P).
	1 kilowatt = 1,000 watts = kw.
Ampere, unit of current or rate of flow, C (I).	Joule, the unit of work, J (W).
	H.P. = horse-power.
Ohm, unit of resistance, R.	t = one second.
Coulomb, unit of quantity, Q.	T = one hour.
Ampere-hour = 3,600Q = Q'.	

Electrical Equations.—Using the above notation the relation between the various units may be expressed by the following equations, which may be transposed in the same manner as any algebraic equation:

$$E = CR \qquad Q = Ct$$
$$C = \frac{E}{R} \qquad Q' = CT$$
$$R = \frac{E}{C} \qquad W = \begin{cases} CE \\ \text{or } C^2R \\ \text{or } \frac{E^2}{R} \end{cases} \qquad J = \begin{cases} QE \\ \text{or } CEt \\ \text{or } C^2Rt \\ \text{or } \frac{E^2t}{R} \end{cases} \qquad \text{H.P.} = \begin{cases} \frac{W}{746} = \frac{\text{kw.}}{.746} \\ \text{or } \frac{C^2R}{746} \end{cases}$$

EXAMPLES SHOWING APPLICATIONS OF ABOVE FORMULAS.

Example 1.—What voltage is required to send a current of 22 amperes through a wire having a resistance of 5 ohms? *Ans.* $E = 22 \times 5 = 110$ volts.

Example 2.—How many amperes will flow through a copper wire having a resistance of 5 ohms, the voltage being 110? *Ans.* $C = \frac{110}{5} = 22$ amperes.

Example 3.—The pressure on a circuit is 110 volts, and it is desired to supply current sufficient for twelve 16-c.p. lamps (6 amperes), what should be the resistance of the circuit? *Ans.* $R = \frac{110}{6} = 18.33$ ohms.

Example 4.—The common 110-volt incandescent lamp has a resistance of about 216 ohms. (1) What current is required with a pressure of 110 volts? *Ans.* $C = \frac{110}{216} = .51$ ampere. (2) How many watts does it consume? *Ans.* $W = CE = .51 \times 110 = 56.1$ watts. (3) How many such lamps can be supplied by 1 electrical H.P.? *Ans.* 1 H.P. = 746 watts. If one lamp requires 56.1 watts, the number of lamps that can be supplied by 746 watts $= \frac{746}{56.1} = 13.3$ lamps. (4) How many such lamps will 10 kw. suffice? *Ans.* 10kw. $= 10 \times 1{,}000$ watts $= 10{,}000$ watts. $\frac{10{,}000}{56.1} = 178$ lamps.

Example 5.—How many H.P. will 10 kw. furnish? *Ans.* H.P. $= \frac{10}{.746} = 13.4$ horse-power.

Dynamo-electric Machines.—There are three classes of dynamo-electric machines, viz.:

1. *Generators* for generating an electric current.

2. *Motors* for converting electrical into mechanical energy.

3. *Rotary converters* for changing the voltage of direct currents, or the voltage, phase, or frequency of alternating currents, and also for changing alternating currents to direct or vice versa, and

3*a*. *Transformers* for converting one voltage into a higher or lower voltage. Converters and transformers belong to the same class.

A motor is the same machine as a dynamo, but with the nature of its operation reversed.

Generators are of two general classes, viz., continuous-current and alternating-current machines; the former are commonly called dynamos and the latter alternators.

Generators and motors of all kinds vary in voltage, current, and speed, according to the purpose for which they are designed.

A *transformer* consists essentially of two coils of wire, one coarse and one fine, wound upon an iron core. Its function is to convert electrical energy from one voltage to another. If it reduces the voltage it is known as a "step-down" transformer, and if it raises it, it is known as a "step-up" transformer.

Kinds of Currents Produced.—There may be said to be four kinds of electrical currents, viz.: (1) Direct currents, constant-potential, or pressure. (2) Direct currents, constant current. (3) Alternating currents, constant-potential. (4) Alternating currents, constant current.

Alternating currents may be single-phase, two-phase, three-phase, five-phase, or any other number, depending upon the number of poles and armature winding of the generator.

A current used for either lighting or power cannot be constant in both pressure and rate.

Both for lighting and power a *constant pressure* is more desirable than a *constant current* with varying pressure.

"A direct current is uniform in strength and direction, while an alternating current rapidly rises from zero to a maximum, falls to zero, reverses its direction, attains a maximum in the new direction and again returns to zero. The advantages of alternating over direct currents are: 1. Greater simplicity of dynamos and motors, no commutators being required; 2. The feasibility of obtaining high voltages by means of transformers for cheapening the cost of transmission; 3. The facility of transforming from one voltage to another, either higher or lower, for different purposes." (Kent, p. 1063.)

Electric Lighting.

Systems Commonly Used for Supplying the Electrical Energy to Lamps.

Direct-current, Constant-potential Systems.— *a. Two-wire system* largely used for incandescent lighting from small plants, as for a large office building or factory; it is usually operated at 110 volts.

b. Three-wire system used in small towns for the lighting of buildings from the public mains, usually operated at 220 volts. Also in large cities with underground conduit system.

The ordinary three-wire system requires two dynamos to balance the load.

Five-wire and seven-wire systems with high voltage have been used in Europe, but very little in America.

Direct-current, Constant-current System.—This system is largely used for municipal and commercial arc lights, but is rarely used for incandescent lighting.

Alternating-current, Constant-potential Systems.—*a. Single-phase System.*—Current transmitted to building at 1,000 to 2,000 volts and reduced to 50 to 110 volts by a transformer.

b. Two-phase System.—Two or three wires; most used for lighting from public plants, principally because it enables both lights and motors to be operated from the public dynamo.

c. Three-phase System.—Three or four wires; used for same purpose as the two-phase system.

All three of these systems are used both for incandescent lighting and power from central stations.

An alternating current may be changed to direct current at a sub-station by a rotary converter.

Alternating-current, Constant-current System, practically if not wholly obsolete.

Fuses, Cut-outs, and Circuit-breakers.—*The fuse* consists of an easily fusible metal, generally a mixture of lead and bismuth, which is inserted in the circuit. The passage of an excesssive or dangerously large current from any cause melt the fuse and breaks the circuit. The cause of the large currents may then be removed and a new fuse inserted in place of the old one.

TABLE I.—RELATIVE WEIGHT OF COPPER REQUIRED IN DIFFERENT SYSTEMS FOR EQUAL EFFECTIVE VOLTAGE (KENT).

Direct-current, ordinary two-wire system	1.000
Direct-current, three-wire system, all wires of same size	.375
Direct-current, three wires, neutral, one-half size	.313
Alternating-current, single-phase two-wire and two-phase four-wire	1.000
Two-phase three-wire, voltage between outer and middle wire same as in single-phase two-wire	.729
voltage between two outer wires same	1.457
Three-phase three-wire	.750
Three-phase four-wire	.333

Cut-outs and circuit-breakers are automatic safety devices required for the protection of all constant-potential systems whatever the voltage. Both are for the purpose of protecting the wires from damage due to the presence of too much current from any cause whatever.

The ordinary cut-out consists of a porcelain base that has suitable terminals for inserting a fuse between the ends of the wire. It must be constructed so that the blowing out of a fuse can do no damage, *i.e.*, set anything on fire, and placed where it can easily be reached to replace the fuse.

Formerly a piece of fuse wire was used in cut-outs, but the underwriters now require enclosed fuses (Fig. 1) or fusible plugs

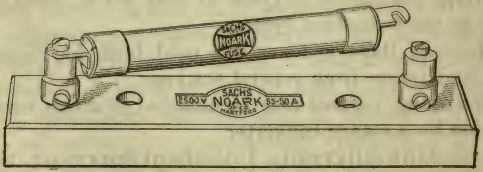


Fig. 1
Enclosed Fuse.

which screw into a receptacle. Fuse plugs may be used for currents up to 30 amperes; above that enclosed fuses must be used. Fuse plugs and enclosed fuses are somewhat more expensive than the link fuse, but are considered safer. A cut-out or circuit-breaker is required at or near the place where the wires enter a building, and every circuit of twelve 16-c.p. lights must be protected by a cut-out.

Circuit-breakers are automatic switches controlled by an electromagnet and are made in a variety of styles.

They are more expensive than fusible cut-outs, and are generally used only on switchboards for large installations and where it is desirable to open the circuit instantly on certain loads, which a fuse cannot be depended on to do with any degree of accuracy, owing to both time and surrounding temperature factors.

Also used largely on installations where the variation in load is large and often and the frequent burning out of fuse would become expensive both for renewals and time required to replace them.

Lamps.—Two kinds of lamps are used for electric lighting—incandescent lamps and arc lamps. The former are used principally for interior illumination, although sometimes used for street lighting, especially where the streets are thickly shaded by trees. Arc lamps are especially adapted for street lighting and for large interiors where they can be kept above the range of the eye, as in railway stations, stores, etc.

Incandescent lamps as commonly made consist of a glass bulb containing a simple carbon conductor the ends of which are connected to the source of the electric current. When the current flows through the carbon filament it heats it to such a degree that it becomes incandescent; hence the name of the lamp.

Voltages.—In order that the current shall cause the lamp to give its rated candle-power, it must be designed for the voltage at which the system is run. If the voltage of the current is much greater than that for which the lamp is designed it will quickly burn out the carbon filament, while if the voltage of the current is below that of the lamp, it will not give its rated candle-power, a voltage 10 per cent. lower reducing the candle-power about one half.

The voltage most commonly used for 16-c.p. lamps is from 104 to 110.

Lamps are also made for voltages of 45 to 250, and 1-c.p. lamps, for illuminating signs or decorative purposes, are made for 12.5 and 15 volts, these lamps being commonly used in series, eight lamps on a 110-volt circuit. Two 4-c.p. lamps, 52 volts, are also often used in series on a 110-volt circuit.

Candle-pouer.—Incandescent lamps of 110 volts are commonly made 4, 8, 10, 12, 16, 24, and 32 candle-power. Table II

shows the standard candle-powers, voltages, and current required for incandescent lamps.

For data pertaining to the Meridian and Nernst lamps, see pp. 1298, 1299.

TABLE II.—INCANDESCENT-LAMP DATA.*

Volts.	Candle-power.	Current, Amperes.	Watts per Lamp.
52	4	.39	20
"	8	.61	32
"	10	.67	35
"	16	1.08	56
"	20	1.34	70
"	24	1.62	84
"	32	2.15	112
104	10	.34	35
"	16	.54	56
"	20	.67	70
"	24	.81	84
"	32	1.08	112
110	8	.27	30
"	10	.32	35
"	16	.51	56
"	20	.64	70
"	24	.76	84
"	32	1.02	112
"	50	1.59	175
"	100	3.18	350
"	150	4.77	525
220	16	.291	64
"	32	.582	128

* H. C. Cushing, Jr., in Practical Lessons in Electricity.

Arc Lamps.—These are of two kinds, *open arc lamps* and *enclosed arc lamps*, the latter being generally used for interior illumination. The light from the enclosed arc is much softer and steadier than that from the old-style open arc, there are no sparks, and the life of the carbon is from twelve to fifteen times as great as in the open arc.

"Current for arc lighting is furnished either on the series constant-current or on the parallel constant-potential system. In the latter the voltage of the circuit is usually 110. In cur-

rents with higher voltages lamps are used in series, for instance 5 to 10 with a 500-volt circuit.

"*Direct-current open arcs* usually require about 10 amperes at 45 volts, or 450 watts. The range of voltage is from 42 to 52 for ordinary constant-current arcs. The most satisfactory light is given by 45 to 47 volts.

"*Alternating-current open arcs* usually take about 15 amperes at 30 to 35 volts, but are not much used. With the same energy and carbons, the mean spherical candle-power is about one half that of the continuous-current open arc.

"*Direct-current enclosed arcs* consume about 5 amperes at 80 volts, or 400 watts. Alternating-current enclosed arcs usually take a current of 6 amperes at 70 or 75 volts." *

Arc lamps generally require a resistance in series with the arc in order to regulate properly. This resistance is usually placed within the structure of the lamp, and is adjustable so that a single lamp can be made to burn well on any circuit from 105 to 120 volts.

Methods of Connecting Lamps.

There are three ways of connecting lamps to the distribution wires, viz.: (1) in series, (2) in parallel, and (3) in parallel series.

Lamps in Series.—Lamps are said to be connected in series when they are arranged one after the other, so that the same current flows through all the lamps.

The lamps shown by Fig. 2 are in series. When conductors are arranged in series the total resistance of the circuit is the

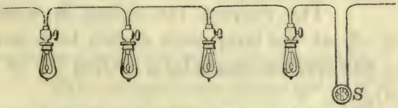


Fig. 2

Lamps in Series.

sum of the resistances of the several parts, and the pressure required to force the current through a number of lamps in series is the sum of the voltages required for the separate lamps. Thus the voltage required to supply the proper current for four 52-volt lamps is $4 \times 52 = 208$ volts. Arc lamps for street

* Kent, p. 1044.

lighting are often connected in series, but incandescent lamps are almost never connected in series except for decorative purposes and in electric signs. Where lamps of low voltage are used on 110-volt systems it is necessary to connect them in series. The underwriters do not approve connecting incandescent lamps in series. The series system requires a *constant current* with varying pressure, and if one lamp burns out the circuit is broken and all of the lamps will go out, unless some provision is made for maintaining the circuit around the lamps.

Lamps in Parallel.—This is the common method of connecting incandescent lamps. It is illustrated by Fig. 3. With this system the pressure in each lamp is the same as in the distributing lines, and any lamp may be turned on or off without affecting the other lamps. For this system the *pressure* or voltage must be kept constant, while the current or quantity of electricity flowing in the lines will depend upon the number of lamps that are burning. Thus with twelve 16-c.p. lamps of 110 voltage on a parallel circuit, each lamp requiring .51 ampere (see Table II), when all the lamps are burning, a current of 6.12 amperes, or 673.2* watts, will be required, but with but one lamp burning, a current of only .51 ampere will flow. The voltage, however, must be the same for one lamp as for the twelve. For lamps in parallel, therefore, a *constant-potential* system is required.

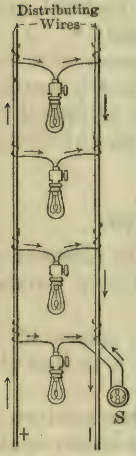


Fig. 3

The current for lamps in parallel may be turned on or off at the lamp, or a switch loop may be run any distance and the contact made by a switch (S) as for the lower lamp (Fig. 3).

Lamps in Parallel Series.—This method is a combination of the other two. Parallel lines are run as in the parallel system, but two or more lamps are connected in series between them as in Figs. 4 and 5. This method of connecting lamps is used principally in places where it is desired to operate lamps on a power system. Fig. 4 shows series of five lamps operated on a 500-volt system, and Fig. 5 series of two lamps on a 110-

* Watts being equal to amperes times voltage.

or 220-volt system using 52- or 110-volt lamps respectively. Any number of series may be connected across the mains, each series being independent of the others. But in each series if one light burns out, the others will go out, and one lamp cannot

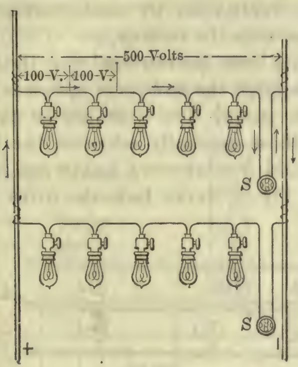


Fig. 4

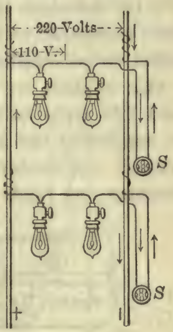


Fig. 5

Lamps in Parallel Series.

be used without using the others. The sum of the voltages of the lamps in series must be approximately equal to the voltage between the mains. There are a number of special cases in which this method of connection may be used.

[*Note.*—Although the lamps in Figs. 3, 4, and 5 are connected directly across the wires, this is not necessary in practice so long as the lamp wires are connected to the distributing wires or mains. Thus five lamps in series on a 500-volt circuit may be connected as in Fig. 6.]

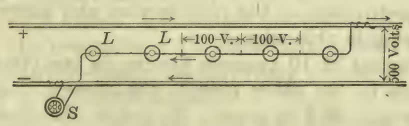


Fig. 6

The Edison Three-wire System.—Figs. 3, 4, and 5 are examples of the two-wire system of distribution, which is the system recommended for average sized office buildings, apartment houses, theatres, and stores.

Where power is to be taken from the same plant and is not

too great a portion of the capacity of the installation this system may also be used, but separate mains should under all circumstances be run for the motors, as the variation in load and consequently the current demand on the mains would cause a very appreciable fluctuation in candle-power of the lamps if on the same mains with the motors.

Where comparatively long lines are required and the amount of current to be supplied is large the *three-wire system* is used.

By this system we can supply two voltages or pressures, 110 and 220 volts being those generally adopted, the 110-volt circuit supplying the arc and incandescent lights and the 220-volt circuit the motors. Fig. 7 shows how the wires are run and connections made.

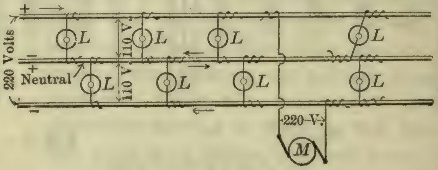


Fig. 7

The pressure between the two outside wires is the full voltage transmitted from the dynamos or transformer, usually 220 volts for interior wiring. The current in these two wires flows in opposite directions. The middle wire, called the neutral wire, forms one side of two circuits, the current from one circuit tending to flow in one direction and that from the other circuit in the opposite direction; consequently when currents of the same strength (in amperes) are flowing in both circuits they neutralize each other in the middle wire and there will be no current flowing in this wire.

With a current of 10 amperes flowing in one circuit and one of 6 amperes in the other circuit, the current flowing in the neutral wire will be 4 amperes. To obtain the greatest benefit from this system, it should always be installed so that there will be nearly the same load or number of lamps on each side of the neutral wire. Even then there will be times when more lamps will be burning on one side than on the other, so that it is necessary to give some size to the neutral wire.

The neutral wire is seldom made less than one half the cross-section of the outer wires. For distributing mains in build-

ings carrying lamps only, the neutral wire should be of the *same size* as the outer wires.

From Table I it will be seen that the three-wire system effects a considerable saving in copper, amounting to fully 60 per cent. of the ordinary two-wire 110-volt system.

As a rule in supplying current for light and power from one plant, the main wires only are arranged on the three-wire system and the distributing wires are run on the two-wire system as in Fig. 8.

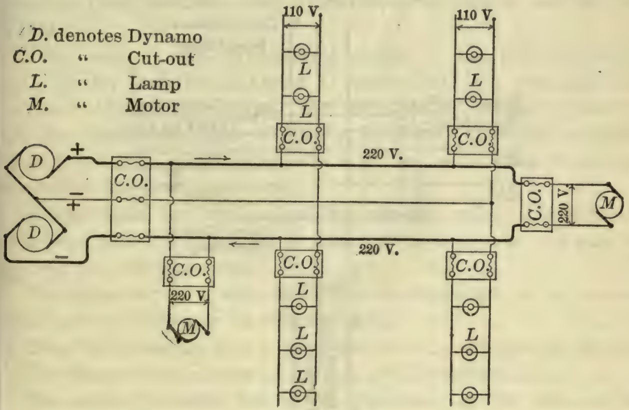


Fig. 8
Example of Three-wire System of Wiring.

When using the three-wire system for lighting only, the three wires are usually run no farther within the building than to the centres of distribution, and from these centres two wires are run for each circuit, the circuits being divided as equally as possible on the two sides of the three-wire system as shown by Fig. 9. Three-wire mains are now very commonly used where the current exceeds 100 amperes.

When motors are operated from the three-wire system they are usually connected only to the outside wires.

Motors used on three-wire incandescent-lighting systems should be wound for 220 volts.

WIRE CALCULATIONS.

Wire Gauges.—As the diameter of wires are ordinarily designated by the numbers of a wire gauge, and as there are a number of wire gauges in common use, some knowledge of those used for copper wire is necessary.

The Brown & Sharp, or B. & S., gauge is almost exclusively used in America in connection with electrical work, except

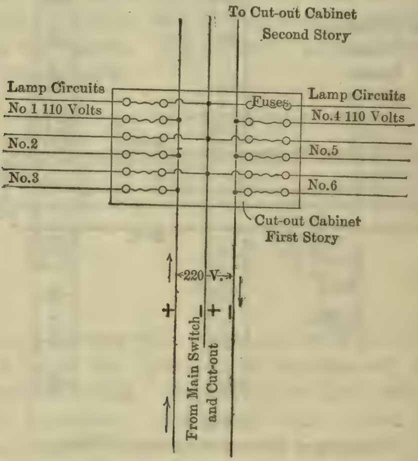


Fig. 9

where the size of the wire is designated in circular mils. The sizes of wire given by this gauge range from No. 0000 (.46 in.) to No. 40 (.0031 in.), but No. 14 is the smallest size permitted for interior wiring. The No. 10 wire has a diameter of very nearly $\frac{1}{10}$ of an inch, and its resistance per 1,000 ft. is very nearly 1 ohm. For any given number of this gauge a wire three numbers higher has very nearly half the cross-section, and one three numbers lower has twice the cross-section; thus a No. 13 wire has very nearly one half the cross-section of a No. 10 wire, and a No 7 has twice the cross-section of a No. 10, or four times that of a No. 13.

The Circular-mil Wire Gauge.—This gauge was designed by the engineering department of the Edison Company

especially for the designation of copper wire for electrical work, and is now in universal use in this country. In practice the B. & S. gauge is commonly used for designating wires up to No. 0 or No. 00, and all wires above that size are designated by circular mils (c.m.).

The size of wire required is often determined in circular mils and designated by the corresponding B. & S. gauge number, which is readily done by means of Table III.

Copper wire is sold by the pound if bare or of the numerous weather-proof varieties, but rubber-covered wire is sold by the 1,000 ft.

The basis of the circular-mil gauge is the area of a wire $\frac{1}{1000}$ of an inch in diameter (1 mil=.001 in.), consequently 1 c.m.=.0000007854 sq. in. As the area of circles is directly as the square of their diameter, it follows that the sectional area of a wire 2 mils in diameter=4 c.m., of a wire 10 mils in diameter 100 c.m., and so on.

When wires are designated by circular mils, the *sectional area* and not the diameter is generally given, c.m. always referring to sectional area.

The diameter of a wire in *mils or in thousands of an inch*= square root of its area in *circular mils*.

Thus the diameter of a wire of 3,600 c.m.=60 mils, or .060 in.

The diameter of a wire 14,400 c.m.=120 mils=.12 in.

The area of a wire .162 in. in diameter, or 162 mils,$=162^2$ $=26,244$.

To reduce circular mils to square inches multiply by 7,854 and point off ten places of decimals. Thus, 5,000 c.m.=7,854 ×5,000=.0039270000 sq. in.

To obtain the sectional area of a square or rectangular bar in circular mils multiply together its dimensions in mils and the product by 1.273.

Example 6.—What is the sectional area in circular mils of a bar $\frac{1}{8}$ in.×$\frac{1}{4}$ in.? *Ans.* $\frac{1}{8}$ in.=.125 in.=125 mils, $\frac{1}{4}$ in.=.250 in. =250 mils; 125×250×1.273=39,781.25 c.m.

The weight of bare copper wire per 1,000 ft.=c.m.×.003027 lbs. Thus the weight of 1,000 ft. of copper wire having a sectional area of 2,000 c.m.=.003027×2,000=6.054000 lbs.

Table IV gives the dimensions and weights of bare copper wire from No. 18 to No. 4–0 B. & S. gauge.

Carrying Capacity of Copper Wire.—The safe carrying capacity of copper wire for interior wiring is practically

fixed by the underwriters, and if the capacity limits given by the table published by them are exceeded it would tend to destroy the right to recover insurance in case of fire.

The safe carrying capacity of rubber-covered and weather-proof wires given by the National Board of Fire Underwriters is shown by Table III.

The lower ampere capacity assigned to rubber-covered wires is due to the fact that the rubber insulation would deteriorate in quality under a temperature as high as that allowed for weather-proof wire; *i.e.*, the rubber covering makes necessary a lower rate of heat development than is required for safety from fire.

No smaller than No. 14 *wire* may be used under insurance rules, except that No. 16 may be used for flexible cord and No. 18 for fixture wiring. Nos. 13, 11, 9, and 7 are not usually carried in stock and can only be purchased on special order.

Rubber-covered wire must be used for service wires, for moulding work, and in damp places; it is more expensive than weather-proof wire. The latter wire may be used in open or exposed places and for outside line wires.

Drop of Potential.—When an electric current flows through a wire of any appreciable length the pressure becomes reduced by the resistance of the wire, so that if the current enters the wire at, say, 110 volts, at the extreme end of the circuit it will be somewhat less, depending upon the length and sectional area of the wire. Drop of potential corresponds to loss of head in hydraulics. As a drop of voltage materially below that for which the lamps are designed means diminished candle-power, it is very important that the wires be proportioned so that the drop shall not be sufficient to affect the illumination.

Mains and distributing wires may be capable of carrying the number of amperes in accordance with Table III, and yet cause a drop of potential of such magnitude that the most distant lamps will burn only at a dull red.

An excessive drop in voltage also means increased cost for light and not enough copper in the wires.

Where the current is supplied from the public mains it is usual to specify a 2 per cent. drop, but where the current is produced cheaply, as by a dynamo on the premises, a 3 per cent. or 5 per cent. drop may be allowed. Not more than a 5 per cent. drop on short distances should be permitted, even where very cheap work is desired.

The drop in volts (not in percentage) = current in line × resistance of line, or drop in volts = amperes × ohms.

Example.—What will be the drop in a circuit of No. 14 copper wire 280 ft. long, supplying nine lamps, requiring 4.5 amperes? *Ans.* From Table V we find that the resistance of No. 14 wire is 2.527 ohms per 1,000 ft., hence for 280 ft. it will be 2.527 × .280 = .7075 ohm, and drop in volts = 4.5 × .7075 = 3.1837 volts. The voltage for this current (.5 ampere per lamp) will be about 110, consequently the percentage of drop $= \frac{3.1837}{110} = 2\frac{9}{10}$ per cent., nearly. Two per cent. drop on a pressure of 110 volts is 2.2 volts.

Centre of Distribution.—The meaning of this term may best be illustrated by an example. Let Fig. 10 represent a circuit

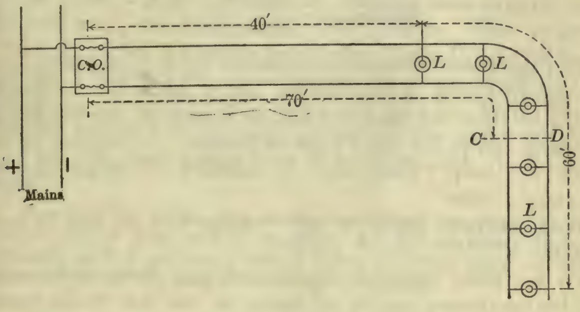


Fig. 10

carrying six lamps, the first lamp being 40 ft. from the cut-out, or source of supply. The whole of the current must be transmitted through this 40 ft., but from that point it will gradually fall off, and the average current will only extend to the point *CD*, half way between the extreme lamps. Or, in other words, the centre of distribution is analogous to the centre of gravity of the lamps on the circuit.

The centre of distribution determines the length of the line in the rules for finding the necessary size of wire.

Distributing centres are the points in a building where the cut-out cabinets are located and the branch circuits taken off.

Calculations for Size of Wire for Incandescent Lighting.—The sizes of wires for interior lighting are or should be always determined on a basis of a fixed drop of potential,

usually 2 volts on the distributing circuit and 2 to 3 volts on the feeders or mains.* The size of wire may be determined either in terms of its sectional area in circular mils or in terms of its resistance in ohms per 1,000 ft.

Knowing the sectional area in circular mils the corresponding gauge number may be found from Table III, or if we have the resistance in ohms per 1,000 ft., we may find the corresponding gauge number from Table IV.

The formula for circular mils is as follows:

$$\text{Circular mils} = \frac{10.8 \times 2d \times N \times c}{v}. \quad . \quad . \quad . \quad (A)$$

The formula for resistance per 1,000 ft. of wire is

$$\text{Resistance} = \frac{1{,}000v}{N \times c \times 2d}. \quad . \quad . \quad . \quad . \quad . \quad (B)$$

In both these formulas d = distance in feet, one way, from cut-out to centre of distribution (see p. 1323) for distributing wires, or from entrance cut-out or source of current to distributing centre for main lines or feeders. c = current in amperes *per lamp* (Table II). N = number of lamps supplied. v = drop in volts.

Both formulas apply to any voltage and to any two-wire system.

To use these formulas for the ordinary three-wire system, let N = maximum number of lamps on *one side* of the neutral wire and *double the drop in volts.* The neutral or middle wire should be of the same size as the outside wires (see top of p. 1319).

Example 7.—The distance from the cut-out to centre of distribution of a circuit carrying twelve 16-c.p. 110-volt lamps is 50 ft. What size of wire should be used for a drop of 2 volts?

Ans. $d = 50$; $N = 12$; c (Table II) $= .51$, and $v = 2$.

By formula (A),

$$\text{Circular mils} = \frac{10.8 \times 100 \times 12 \times .51}{2} = 3{,}305.$$

From Table III, we see that the next larger size of wire is 4,107 c.m., equivalent to a No. 14 wire.

By formula (B),

$$\text{Resistance per 1,000 ft.} = \frac{1{,}000 \times 2}{12 \times .51 \times 100} = 3\ 268,$$

* Many municipal lighting companies require that there shall be no more than 2 per cent. total drop in the wiring for interior lighting.

which we see from Table IV is about the resistance of a No. 15 wire, but as No. 14 is the smallest wire permitted we must use that size.

Example 8.—The distance from the entrance cut-out (where the wires enter the building) to the main distributing centre of a building is 100 ft. The total number of 16-c.p. 110-volt lamps supplied is ninety. What size mains should be used on the two-wire system with a drop of two volts?

Ans. $d=100$; $N=90$; $c=.51$; $v=2$.

By formula (A),

$$\text{Circular mils}=\frac{10.8\times200\times90\times.51}{2}=49{,}572.$$

Looking in Table III, we see that we must use No. 3 wire. If we allow a drop of 3 volts the sectional area required will be 33,048 c.m., which requires a No. 5 wire. The weight per 1,000 ft. of No. 3 weather-proof wire (Table IV) is 200 lbs. and of No. 5 wire 125 lbs., consequently the *saving in weight of wire* by using a drop of 3 volts instead of 2 is 75 lbs., or $37\frac{1}{2}$ per cent. of 200, and as wire is sold by the pound, the *saving in cost* with a 3 per cent. drop ranges from 30 to 40 per cent. of a 2 per cent. drop.

Example 9.—With the same conditions as given in Ex. 8, what size of wire will be required for the ordinary three-wire system with 2 per cent. drop? *Ans.* In this case we use one half of N, or 45, and $2v$ instead of v; then

$$\text{Circular mils}=\frac{10.8\times200\times45\times.51}{4}=12{,}392,$$

or just *one fourth* the section required for the two-wire system. The size of wire required is No. 8 (a No. 9 would answer if it could be had). Comparing the weight of wire required with the two-wire system, we have two No. 3 wires weighing 400 lbs. per 1,000 ft., and with the three-wire system three No. 8 wires weighing 207 lbs., hence the saving in cost is nearly 50 per cent., and if No. 9 wire were obtainable the saving would be 55 per cent.

With a drop of 3 per cent. (3.3 volts) the circular mils required for the three-wire system $=\dfrac{10.8\times200\times45\times.51}{6.6}=7{,}510$, requiring No. 10 wires.

The current in amperes in the two-wire system $=N\times c=45.9$, and in the three-wire system $\frac{1}{2}N\times c=22.95$.

Referring to Table III, we see that the smallest size of weather-proof wire permitted for 45.9 amperes is No. 8; consequently we could use No. 8 wire with the two-wire system

and comply with the underwriters' rules, but the drop in potential would be $45.9 \times .2 \times .6285$ (amperes × resistance of line) $= 5.77$ volts, or over 5 per cent.

TABLE III.—CARRYING CAPACITY OF WIRES AND CABLES.

For interior conductors, all voltages.

(From the National Electrical Code.)

Wires, No. B. & S. Gauge.	Circular Mils.	Capacity in Amperes.	
		Rubber-covered.	Weather-proof.
18	1,624	3	5
16	2,583	6	8
14	4,107	12	16
12	6,530	17	23
10	10,380	24	32
8	16,510	33	46
6	26,250	46	65
5	33,100	54	77
4	41,740	65	92
3	52,630	76	110
2	66,370	90	131
1	83,690	107	156
0	105,500	127	185
00	133,100	150	220
000	167,800	177	262
0000	211,600	210	312
Cables	200,000	200	300
"	300,000	270	400
"	400,000	330	500
"	500,000	390	590
"	600,000	450	680
"	700,000	500	760
"	800,000	550	840
"	900,000	600	920
"	1,000,000	650	1,000
"	1,100,000	690	1,080
"	1,200,000	730	1,150
"	1,300,000	770	1,220
"	1,400,000	810	1,290
"	1,500,000	850	1,360
"	1,600,000	890	1,430
"	1,700,000	930	1,490
"	1,800,000	970	1,550
"	1,900,000	1,010	1,610
"	2,000,000	1,050	1,670

A current of one ampere will supply two 16-c.p. lamps.

For the three-wire system, the current being 23 amperes, the smallest weather-proof wire permitted by Table III is No. 12, which would give a drop of 7.4 volts, or 3.8 volts on each side, or about 3½ per cent. of the lamp voltage. Except on very short lines a 2 per cent. drop will always demand larger wires than required by the underwriters, and this is also usually true of a 3 per cent. drop.

TABLE IV.—DIMENSIONS, WEIGHT, AND RESISTANCE OF COPPER WIRE.

Gauge No. B. & S.	Diam. in Mils.	Area in Cir. Mils.	Area in Sq. Ins.	Weight in Lbs. per 1,000 Feet.		Resistance in Ohms † per 1,000 Ft.
				Bare Wire.	Weather-proof * Wire.	
0000	460	211,600	.166190	640.73	800	.04904
000	410	167,800	.131790	508.12	666	.06184
00	365	133,100	.104520	402.97	500	.07797
0	325	105,500	.082887	319.74	363	.09827
1	289	83,690	.065732	253.43	313	.12398
2	258	66,370	.052128	200.98	250	.15633
3	229	52,630	.041339	159.38	200	.19714
4	204	41,740	.032784	126.40	144	.24858
5	182	33,100	.025999	100.23	125	.31346
6	162	26,250	.020618	79.49	105	.39528
7	144	20,820	.016351	63.03	87	.49845
8	128	16,510	012967	49.99	69	.62849
9	114	13,090	.010283	39.65	....	.79242
10	102	10,380	.008155	31.44	50	.99948
11	91	8,234	.006466	24.93	....	1.2602
12	81	6,530	.005129	19.77	31	1.5890
13	72	5,178	.004067	15.68	....	2.0037
14	64	4,107	.003225	12.44	22	2.5266
15	57	3,257	.002558	9.86	....	3.1860
16	51	2,583	.002028	7.82	14	4.0176
17	45	2,048	.001608	6.20	....	5.0660
18	40	1,624	.001275	4.92	11	6.3880

* Approximate weight of weather-proof line wire for outdoor work is 10 per cent. less than here given.

† Values given by H. C. Cushing, Jr., in Practical Lessons in Electricity. The author has been unable to find any two tables that give exactly the same resistance.

To find the smallest size of wire that will comply with the underwriters' rules it is only necessary to compute the total current in amperes, and from Table III select the wire having

a capacity equal to or next above the required number of amperes. Table VI shows at a glance the maximum number of 16 c.p. 110 volt lamps permitted by the National Code.

TABLE V.—MAXIMUM LENGTH OF LINE FOR GIVEN NUMBER OF LAMPS THAT CAN BE USED WITH A 2 PER CENT. DROP. TWO-WIRE SYSTEM.

Based on ½ ampere per lamp. One 32-c.p. lamp = two 16-c.p. lamps Two 24-c.p. lamps = three 16-c.p. lamps.

No. of Wire, B. & S. Gauge.	Number of 16-c.p. 110-volt Lamps.								
	4	6	8	10	11	12	16	20	24
	Maximum Length of Line, One Side, in Feet.								
14	209	139	104	83	76	70	52	42	35
12	.....	221	166	133	120	110	83	66	55
10	.....	.....	264	211	192	176	132	105	88
8	.....	.....	.....	326	297	272	204	163	136
6	.....	.....	.....	.....	.....	440	334	267	220
	Number of 16-c.p. 110-volt Lamps.								
	30	36	40	50	60	70	80	90	100
	Maximum Length of Line, One Side, in Feet.								
12	44	37							
10	70	58	52	42					
8	109	91	81	65	54	37	40		
6	178	148	133	107	89	76	66	59	53
5	225	187	168	135	112	96	84	75	67
4	.....	236	212	170	141	121	106	94	85
3	.....	.....	268	214	180	153	134	119	107
2	.....	.....	.....	270	225	193	169	150	135
1	.....	.....	.....	.....	285	243	213	190	170

For three-wire mains with 220 volts between outer wires and same number of lamps on each side length of wire may be increased four times.

Formulas (A) and (B) may also be used for *motor wiring*, if the required current in amperes is known, by substituting the given number of amperes for $N \times c$.

Example 10.—What size of wires should be run to a motor that requires 30 amperes and 220 volts and is situated 200 ft.

from the distributing pole, the drop in volts not to exceed 2 per cent.? *Ans.* Using formula (*A*), and substituting 30 for $N \times c$, we have

$$\text{Circular mils} = \frac{10.8 \times 400 \times 30}{4.4} = 29{,}454,$$

which requires a No. 5 wire.

The current either in watts or amperes is stamped on every motor. If watts are given, the current in amperes may be found by dividing the watts by the voltage. If kilowatts are given, multiply by 1,000 and then divide by the voltage.

TABLE VI.—MAXIMUM CARRYING CAPACITY OF WIRES IN TERMS OF 16-C.P. 110-VOLT LAMPS, HOWEVER SHORT THE WIRES MAY BE.

Based on ½ ampere per lamp.

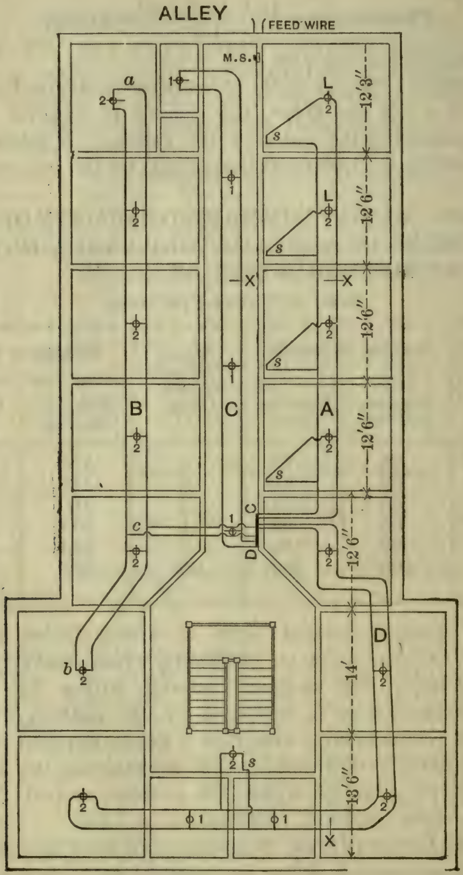
No. of Wire, B. & S. Gauge.	Number of Lamps.		No. of Wire, B. & S. Gauge.	Number of Lamps.	
	Rubber-covered.	Weather-proof.		Rubber-covered.	Weather-proof.
14	24	32	4	130	184
12	34	46	3	152	220
10	48	64	2	180	262
8	66	92	1	214	312
6	92	130	0	254	370
5	108	154	00	300	440

Wiring Tables.—Several forms of wiring tables are published in various books on electricity which are very useful to electricians. For ordinary interior wiring for 110-volt 16-c.p. lamps, Table V, computed by the author, will show at a glance the number of wire, B. & S. gauge, required to supply the given number of lamps by first ascertaining the length of line (one way) through which the average current flows, as explained under Centre of Distribution (p. 1323).

Simple Example of Wiring.—To show the method of wiring an ordinary building for incandescent lighting we will take a two-story building having a floor plan as shown by Fig. 11. The light outlets are all on the ceiling and are indicated by a small circle and cross. The numbers 1 and 2 beside the outlets denote the number of lamps to the outlet.

Current to be obtained from the wires of the public lighting company, which carry a current of 220 volts. The feed-wires

for the building should enter through the alley wall at about the level of the second floor and should drop in the partition

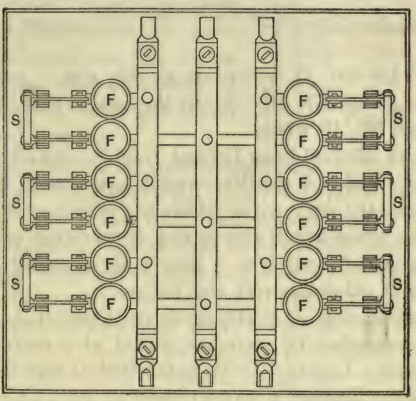


STREET FRONT
SECOND FLOOR PLAN
Fig. 11
Example of Wiring.

just inside the wall for the main fuse block and switch (*M.S.*), which should be in a small cabinet. The distribution cabinet

should be located near the centre of the building, say at *DC*, and there should be a cabinet in each story. From this cabinet we will run four circuits for each story, which are indicated by the letters *A*, *B*, *C*, and *D*. Circuit *A* shows the wires run for a switch on the wall of each of four rooms to control the lights in those rooms. All of the lights on circuit *C* should be controlled by a switch in the cabinet. The lights on circuits *B* and *D* are not switched, except the outlet at head of stairs, which is controlled by a snap or push-button switch at *S*.

For a first-class job all of the four circuits would be controlled by knife switches in the cabinet, as shown by Fig. 12; but this is not absolutely necessary.



F.F, FUSE PLUGS S.S, KNIFE SWITCHES

Fig. 12

Size of Wires.—The centre of distribution of circuits *A*, *C*, and *D* would be at about the points marked *X*. For circuit *B* take one half the distance *ab* and add to it the distance from *c* to the cabinet.

In figuring the length of line, 6 ft. should be added for the drop from ceiling to the cabinet. The number of lamps and length of wire for each circuit are as follows:

Circuit *A*, 8 lights, 41 ft. one way to centre of distribution.
Circuit *B*, 12 lights, 52 ft. " " " " " "
Circuit *C*, 4 lights, 37 ft. " " " " " "
Circuit *D*, 12 lights, 59 ft. " " " " " "
Total number of lamps, 36.

From Table V we see that the maximum length of line one way for No. 14 wire carrying twelve lamps is 70 ft., consequently all of the lamp circuits can be No. 14 wire, which is the smallest size permitted.

Feed-wires.—These should be run on the three-wire system. Allowing for seventy-two lamps in first and second stories and eight in basement, the feed-wires must be capable of supplying eighty lamps. The distance from outside the building to distribution cabinet is about 72 ft., allowing for three drops.

Using formula (*A*), and assuming that there will be forty lamps on each side of the three-wire system, and doubling the drop in volts, we have

$$\text{Circular mils} = \frac{10.8 \times 144 \times 40 \times .51}{4} = 7{,}932 \text{ c.m.},$$

which calls for No. 11 wire; but as this size is not carried in stock we must use No. 10. From the second story to the third we could use No. 12 wires.

For almost all buildings lighted from a central station the lamp circuits will not usually require larger than No. 14 wire, so that about the only wires which the architect needs to look after are the wires which run to the distribution cabinets.

Switches.—A switch is a device for opening or closing a circuit at will other than at the fixture.

In the better class of buildings most if not all of the ceiling lights are controlled by switches placed at a convenient place on a side wall. Lights may be controlled at any distance from the fixture by running a switch loop.

For controlling either a single lamp or fixture, or any number of lamps, a switch loop is run as shown on circuits *A* and *C*, Fig. 10, also by Fig. 3; one side of the loop must be connected with one of the distributing wires and the other side to the lamp.

When a number of lamps are to be controlled by one switch, as in the case of hall lights, and the lamps in large rooms, such as churches, theatres, concert halls, etc., a separate circuit is usually run for those lamps, and a switch anywhere in one of the distributing lines will turn on or off all of the lights.

As the underwriters do not permit more than twelve 16-c.p. lamps on one circuit, not more than twelve lamps can be controlled by one switch, except where the switch is placed on the mains.

It is also practicable to control one lamp from two or three places. Thus by a duplex or three-point switch and proper wiring, a lamp may be lighted or turned off from either the first or second story at will. By means of two three-point switches and one four-point switch a first-story hall lamp may be controlled at will from either the first, second, or third stories. Fig. 13 shows one method of wiring for controlling a hall light from first and second stories. With the switches in the position shown the circuit is broken, as there is no connection between the lamps and line B. By turning either switch a connection is made with line B and the current will flow.*

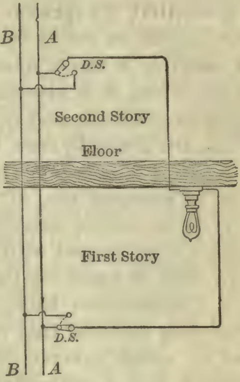


Fig. 13

Kinds of Switches.—For controlling lamps from one point three kinds of switches are used, viz., snap, push-button, and knife switches. When less than eight lamps are controlled by the switch, a push-button switch is commonly used where a neat appearance is desirable, and in places where this is of no importance, a snap switch is used, as it is the cheaper.

Where a circuit of twelve or more lamps is controlled by a switch, a d.p. (double pole) knife switch (Fig. 14) is commonly used, being generally placed in a cabinet.

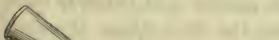


Fig. 14
Common Knife Switch.

Knife switches should always be used on main wires. Snap and push-button switches are made both single and double pole. A *single-pole* switch opens only one side of the circuit and a *double-pole* switch both sides.

A d.p. knife switch necessarily opens both sides, and when used on a three-wire system it must have three poles.

Double-pole snap and push-button switches are seldom used for less than twelve lamps.

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* For method of wiring for controlling lamps from three or more points see p. 41, § 27, vol. 13, International Library of Technology.

Duplex switches (sometimes called three-point switches) are usually of the push-button type.

Conduit Systems.—As weather-proof or rubber-covered wire cannot be run in brick walls or floors of brick, terra-cotta, or concrete without some protection other than the covering of the wires, it is necessary in such places to run the wires in tubes or conduits, and in fireproof buildings all of the lighting wires are generally run in a system of conduits.

Kinds of Conduits.—There are five kinds of interior conduits now in common use, viz.:

1. *Brass-covered conduit*, which is made of paper wound to form a tube, coated with tar on the inside, and covered with a thin shell of brass on the outside.

2. *Circular-loom* tube, a flexible woven tube treated with insulating material that makes it hold its shape. Although it has no metal covering, it is stronger than the brass-covered conduit and is more convenient to use.

3. *Unlined iron pipe.*

4. *Lined iron pipe.*

5. *Flexible armored conduit*, made of metal ribbon wound spirally.

For regular conduit systems only iron piping of the same thickness as ordinary gas piping is approved by the underwriters.

The circular-loom and flexible-steel conduit may be used in dry places and for outlets through plaster if it extends back to the nearest porcelain knob holding the wire which the conduit covers.

The brass-covered conduit was at one time extensively used, but its use is now confined principally to protecting exposed risers on dry walls.

Unlined iron pipe must be galvanized, coated, or enamelled on the inside; lined iron pipe must have an insulating lining $\frac{1}{32}$ in. thick firmly secured to the pipe.

Iron conduit whether lined or unlined is installed in the same manner as a good job of gas fitting, except that for conduits the pipe may be bent to a curve and no elbow can be used having less than $3\frac{1}{2}$-in. radius for the inner edge. Wherever branches are taken off, junction boxes must be provided, and every outlet must have an approved outlet box or plate.

National Electrical Code.—The National Board of Fire Underwriters, in conjunction with committees from the national associations of architects, electrical, mechanical, and railway engineers, have prepared a code of rules and requirements for the installation of electrical lighting which is the generally recognized standard and with which all interior wiring must comply if it is desired to obtain insurance on the building. This code has also been made a part of the ordinances of most of the larger cities.

The National Board of Underwriters also publish, semi-annually, a *supplement* to the National Electrical Code which contains a list of all articles that have been examined and approved for use in connection with the code, together with the names of the manufacturer. Articles not included in this list will not be passed by the inspectors. Copies of the code and supplement can be obtained from the nearest Underwriters' Inspection Bureau, or by writing to the Underwriters Laboratories, 67 East 21st Street, Chicago. The following requirements apply to almost every installation, and every architect should be conversant with them.

Extracts from the National Electrical Code.*

1. All wire for concealed work must be of the best approved rubber-covered brands, as shown in List of Fittings. No wire smaller than No. 14 B. & S. gauge to be used.

2. Where wires are concealed and run parallel to joists they must be supported on porcelain knobs which hold the wires at least 1 in. from woodwork or surface wired over. Knobs must be *securely fastened* and *must be placed every* 4½ *ft. apart.* Where wires are run through joists they must be bushed with porcelain tubes the entire width of joists. All wires must be drawn tight, so as to have all slack removed.

3. In concealed work all wires *must be separated from each other by at least* 5 *ins.* Where wires run down partitions, especially partitions formed by 2×4 studding, the wires must be so supported as to run in centre of partition. If more than two wires are run down partition between studs, they must be separated by at least 5 ins.

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* The numbers here given do not correspond with those in the code, and several of the rules are much abridged. They are intended to give the substance, rather than the exact language.

4. Where wires pass through floors they must be bushed with porcelain tubes and have a floor tube or additional bushing taped securely in place at floor.

5. All joints must be securely soldered and taped. A splice to be approved must be the regular W. U. telegraph joint and must have at least five turns of wire on each side where they join. Joints to be properly taped require where rubber-covered wire is used, first to be taped with rubber tape and then with friction tape.

6. Where wires enter the building they must be provided with a drip loop.

7. *There must be a main cut-out and switch installed in an easily accessible place,* as near as possible to point where wires enter building. (This will require that cut-out and switch be placed where there is no need of a 12-ft. ladder to reach them.)

8. Every lighting circuit of 660 watts must be protected by a cut-out. This will limit the number to twelve 16-c.p. lights on a two-wire 110-volt circuit, or twenty 16-c.p. lights on a three-wire 220-volt circuit.

9. *All cut-outs must be placed in an asbestos-lined cabinet.* Asbestos to be at least $\frac{1}{8}$ in. in thickness and securely held in place by shellac and tacks. Lumber of which cabinet is made must be at least $\frac{3}{4}$ in. in thickness. Cabinet must be furnished with snug-fitting door; door to be hung by strong hinges and to be furnished with a suitable catch.

10. Cut-outs to be approved must be of the plug and cartridge type.

11. Enclosed arc lamps and incandescent lamps must not be placed on same circuit. Arcs must be on separate circuits by themselves. Each arc light must be protected by an approved cut-out. Cut-out to be placed in an asbestos-lined cabinet.

12. The practice of using fused rosettes will not be approved.

13. Where wires run down side wall they must be protected from mechanical injury.

14. All outlets must be made to conform to rule 22 *e*, p. 24, National Electrical Code.

15. Fans or lights in series will not be approved.

16. Runs of lamp cord will not be approved. Lamp cord is designed to be used for drops only. Ordinary insulated wire must be run to place desired.

General Suggestions.

Preface to the National Electrical Code.

In all electric work conductors, however well insulated, should always be treated as bare, to the end that under no conditions, existing or likely to exist, can a grounding or short circuit occur, and so that all leakage from conductor to conductor, or between conductor and ground, may be reduced to the minimum.

In all wiring special attention must be paid to the mechanical execution of the work. Careful and neat running, connecting, soldering, taping of conductors, and securing and attaching of fittings, are specially conducive to security and efficiency, and will be strongly insisted on.

In laying out an installation, except for constant-current systems, the work should, if possible, be started from a centre of distribution, and the switches and cut-outs, controlling and connected with the several branches, be grouped together in a

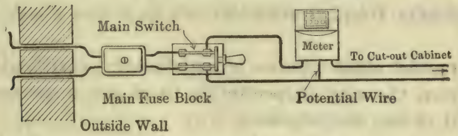

Fig. 15

safe and easily accessible place, where they can be readily got at for attention or repairs. The load should be divided as evenly as possible among the branches, and all complicated and unnecessary wiring avoided.

The use of wireways for rendering concealed wiring permanently accessible is most heartily indorsed and recommended; and this method of accessible concealed construction is advised for general use.

Architects are urged, when drawing plans and specifications, to make provision for the channelling and pocketing of buildings for electric-light or power wires, and in specifications for electric gas lighting to require a two-wire circuit, whether the building is to be wired for electric lighting or not, so that no part of the gas fixtures or gas piping be allowed to be used for the gas-lighting circuit.

Fig. 15 shows a common arrangement of main cut-out, switch, and metre, to comply with rule 7, p. 1336. The main cut-out and switch should be as near as possible to the outside wall, but the metre may be at some distance from the switch if desirable for any reason.

Specifications for Interior Wiring.

Specifications for interior wiring should provide:

1. That the wiring shall be installed in accordance with the latest rules and requirements of the National Board of Fire Underwriters, the local ordinances, and the rules of the local electric light company, where current is to be taken from the public mains.

2. No electrical device or material of any kind to be used that is not approved by the Underwriters' National Electric Association, and all articles must have the name or trade mark of the manufacturer and the rating in volts and amperes or other proper units marked where they may readily be observed after the device is installed.

Requirements 1 and 2 are sufficient to insure a *safe* installation.

3. Contractor must obtain a satisfactory certificate of inspection from the city inspector or from the inspector of the local board of fire underwriters.

4. If the wires are to run in a conduit system it should be so specified. When a conduit system is used, *the wires should not be drawn in* until after the plastering is dry.

5. *Size of Wires.*—The best method is to specify the size of all wires, no wire to be less than No. 14 B. &. S gauge, but if the architect does not care to do this, the following clause is sufficient, provided he can have confidence that the contractor will comply with it: "All wires must be of such size that the drop in potential at farthest outlet shall not exceed 2% under maximum load."

(Wiring specifications for buildings having their own generating plant should be prepared by an expert.)

6. Cut-out cabinets and where they are to be placed; also location of main-line cut-out and fuse.

(For buildings containing more than forty lights, one distributing point is generally sufficient, although in large houses it is often convenient to have a cut-out cabinet on each floor.)

7. Number and kind of switches. All outlets should be marked on the plans, and the number of lights indicated by figures 1, 2, 3, 4, etc., as on Fig. 11. The location of all switches for controlling lights should also be indicated on the plans.

Approximate Cost of Wiring for Incandescent Lighting.—Approximate estimates of the cost of wiring buildings for electric lighting are usually based on the number of *outlets* (not lamps). The actual cost will depend upon the number of pounds of wire required, the kind and number of switches, character of cut-out cabinets, etc., and the time required to do the work, so that a close estimate cannot be made without plans and specifications. Again, wages and prices of material vary to a considerable extent in different portions of the country, so that an estimate that would be about right for one locality would not suffice for another.

The following figures, however, will enable any one to form an approximate idea of what any proposed wiring job will cost.

For new houses of less than seventeen outlets or twenty-five lamps, with no switches except main switch and a rough cut-out box lined with asbestos, allow $1.50 per outlet.

For same class of work, 25 to 100 lamps, allow $1.75 to $2.00 per outlet.

The extra labor involved in wiring old buildings will add from 10 to 50 per cent. to the above figures.

For each switch loop with a single-pole snap switch add $1.50 to $1.75.

For each switch loop with single-pole push-button switch add $2.25 to $2.50.

For each lamp controlled by duplex switches add $5 to $6.

For each hardwood cut-out cabinet with door and lock add from $7 up according to number of circuits and finish.

Iron cut-out cabinets cost from $8.50 up.

Ordinary exposed wiring, as in factories, can usually be run for $1.00 to $1.75 per drop, including rosettes, cord, and sockets, the cost depending very largely upon how closely the drops are spaced.

Small installations with iron-armored conduit will probably cost from $5 to $6 per outlet. Large installations will cost somewhat less.

A private lighting plant of 200 lamps, wired on the concealed knob and tube system, will cost from $1250 to $1500, and a similar plant with 600 lamps will cost from $2500 to $3000

These prices include engine, dynamo-switchboard, etc., complete, and wiring, but no switches for controlling lamps.

The iron-armored conduit system will add about $2.75 per outlet.

None of the above estimates include the cost of fixtures except in the case of exposed wiring.

Drop cord and sockets cost about 90 cts. per lamp. Single-lamp fixtures may be purchased from $1.25 upwards; double-lamp fixtures from $2.00 upwards. Combination fixtures cost about 25 per cent. more than straight electric fixtures.

The price of rubber-covered wire varies from $8.00 to $60.00 per 1,000 ft. according to size, and of weather-proof wire from 16 cts. to 25 cts. per pound.

SPECIFIC GRAVITIES AND WEIGHTS OF VARIOUS SUBSTANCES.*

The Basis for Specific Gravities is Pure Water at 62° Fahr., Barometer 30 Inches. Weight of 1 Cubic Foot, 62.355 Pounds.	Average Sp. Gr. Water = 1.	Average Weight of 1 Cu. Ft., Pounds.
Air, atmospheric at 60° F., under pressure of one atmosphere, or 14.7 lbs. per sq. in., weighs $\frac{1}{815}$th as much as water	.00123	.0765
Aluminum	2.6	162
Anthracite, 1.3 to 1.84; of Penn., 1.3 to 1.7	1.5	93.5
" broken, of any size, loose		52 to 56
" " moderately shaken		56 to 60
" " heaped bushel, loose, 77 to 83 lbs.		
" " a ton loose occupies 40 to 43 cu. ft.		
Antimony, cast	6.70	418
" native	6.67	416
Ash, perfectly dry (see note p. 1344)	.752	47
" American white dry (see note p. 1344)	.61	38
Ashes of soft coal, solidly packed		40 to 45
Asphaltum, 1 to 1.8	1.4	87.3
Brass (copper and zinc), cast, 7.8 to 8.4	8.1	504
" rolled	8.4	524
Brick, best pressed		150
" common and hard		125
" soft inferior		100
Brickwork, pressed brick, fine joints		140
" medium quality		125
" coarse, inferior, soft		100
" at 125 lbs. per cubic foot, 1 cu. yd. equals 1.507 tons and 17.92 cu. ft. equal 1 ton		
Bronze, copper, 8, tin 1 (gun-metal)	8.5	529
Cement, hydraulic. American, Rosendale, " ground, and loose (see p. 192)		56
" hydraulic. Portland, loose (see p. 197)		85 to 90
Charcoal of pines and oaks		15 to 30
Chalk	2.5	156
Cherry, perfectly dry (see note p. 1344)	.672	42
Chestnut, perfectly dry (see note p. 1344)	.660	41
Clay, potters', dry, 1.8 to 2.1	1.9	119
" dry in lump, loose		63

* The values in this table are taken largely from a table compiled by the Cambria Iron Co.

SPECIFIC GRAVITIES AND WEIGHTS OF VARIOUS SUBSTANCES.—*Continued.*

The Basis for Specific Gravities is Pure Water at 62° Fahr., Barometer 30 Inches. Weight of 1 Cubic Foot, 62.355 Pounds.	Average Sp. Gr. Water = 1.	Average Weight of 1 Cu. Ft., Pounds.
Coal, anthracite; see Anthracite.		
" bituminous, solid, 1.2 to 1.5.	1.35	84
" bituminous, solid, Cambria Co., Pa., 1.27–1.34.	....	79 to 84
" bituminous, broken, of any size, loose.	....	47 to 52
" bituminous, moderately shaken.	....	51 to 56
" bituminous, a heaped bushel, loose, 70 to 78.		
" bituminous, 1 ton occupies 43 to 48 cubic feet.		
Coke, loose, good quality.	....	23 to 32
" loose, a heaped bushel, 35 to 42.		
" 1 ton occupies 80 to 97 cu. ft.		
Corundum, pure, 3.8 to 4.	3.9	
Copper, cast, 8.6 to 8.8.	8.7	542
" rolled, 8.8 to 9.	8.9	555
Cork, dry (see note p. 1344).	.24	15
Cypress, American (see note p. 1344).	.55	34.3
Earth, common loam, perfectly dry, loose	....	72 to 80
" " " perfectly dry, shaken	....	82 to 92
" " " perfectly dry, rammed.	....	90 to 100
" " " slightly moist, loose	....	70 to 76
" " " more moist, loose.	....	66 to 68
" " " more moist, shaken	....	75 to 90
" " " more moist, packed	....	90 to 100
" " " as soft flowing mud	....	104 to 112
" " " as soft flowing mud well pressed	....	110 to 120
Elm, perfectly dry (see note p. 1344).	.56	35
Flint.	2.6	162
Glass, 2.5 to 3.45.	2.98	186
" common window.	2.52	157
Gneiss, common, 2.62 to 2.76.	2.69	168
" in loose piles.	....	96
Gold, cast, pure or 24 karat.	19.258	1204
" pure, hammered.	19.5	1217
Granite, 2.56 to 2.88.	2.72	170
Greenstone, trap, 2.8 to 3.2.	3.00	187
Gypsum, plaster of Paris, 2.24 to 2.30.	2.27	141.6
Hay, loose.		
" in stacks, about 512 cu. ft. to ton.		
Hemlock, perfectly dry (see note p. 1344).	.4	25
Hickory, " " " " "	.85	53
Ice, .917 to .922	.92	57.4

SPECIFIC GRAVITIES AND WEIGHTS OF VARIOUS SUBSTANCES.—*Continued.*

The Basis for Specific Gravities is Pure Water at 62° Fahr., Barometer 30 Inches. Weight of 1 Cubic Foot, 62.355 Pounds.	Average Sp. Gr. Water = 1.	Average Weight of 1 Cu. Ft., Pounds.
Iron, cast, 6.9 to 7.4	7.15	446
" gray foundry, cold	7.21	450
" " " molten	6.94	433
" wrought	7.69	480
Lead, commercial	11.38	709.6
Lignum-vitæ (dry)	.65–1.33	41 to 83
Limestone and marbles	2.6	164.4
Lime, quick	1.5	95
" quick, ground, well shaken, per struck bushel, 80 lbs.	....	64
" quick, ground, thoroughly shaken, per struck bushel, 93¾ lbs.	....	75
Locust, dry (see note p. 1344)	.71	44
Mahogany, Spanish, dry (see note p. 1344)	.85	53
" Honduras, dry (see note p. 1344)	.56	35
Maple, dry (see note p. 1344)	.79	49
Marbles (see Limestone).		
Masonry of granite or limestones, well-dressed	....	165
" of granite, well-scabbled mortar rubble; about ⅕ of mass will be mortar	....	154
" of gr'nite, well-scabbl'd dry rubble	....	138
" of granite, roughly scabbled mortar rubble; about ¼ to ⅓ of mass will be mortar	....	150
" of granite, scabbled dry rubble	....	125
" of sandstone, ⅛ less than granite		
Masonry of brickwork (see Brickwork).		
Mercury at 32° Fahr.	13.62	849
Mica, 2.75 to 3.1	2.93	183
Mortar, hardened, 1.4 to 1.9	1.65	103
Mud, dry, close	....	80 to 110
" wet, moderately pressed	....	110 to 130
" " fluid	....	104 to 120
Oak, live, perfectly dry, .88–1.02 (see note p. 1344)	.95	59.3
" white, perfectly dry, .66 to .88 (see note p. 1344)	.77	48
" red, black, perfectly dry	....	32 to 45
Petroleum	.878	54.8
Pine, white, perfectly dry, .35 to .45 (see note p. 1344)	.40	25

SPECIFIC GRAVITIES AND WEIGHTS OF VARIOUS SUBSTANCES.—*Continued.*

The Basis for Specific Gravities is Pure Water at 62° Fahr., Barometer 30 Inches. Weight of 1 Cubic Foot, 62.355 Pounds.	Average Sp. Gr. Water = 1.	Average Weight of 1 Cu. Ft., Pounds.
Pine, yellow, Northern, perfectly dry, .48 to .62 (see foot-note)	.55	34.3
" yellow, Southern, perfectly dry, .64 to .8 (see foot-note)	.72	45
Pitch	1.15	71.7
Poplar, dry (see foot-note)	.47	29
Platinum	21.5	1342
Quartz	2.65	165
Rosin	1.10	68.6
Salt, coarse (per struck bushel, Syracuse, N. Y., 56 lbs.)	....	45
Sand, of pure quartz, perfectly dry and loose	....	90 to 106
" " " " voids full of water	....	118 to 129
" " " " very large and small grains, dry	....	117
Sandstone, 2.1 to 2.73, 131 to 171	2.41	151
" quarried and piled, 1 measure solid makes $1\frac{3}{4}$ (about) piled	....	86
Snow, fresh fallen	....	5 to 12
" moistened, compacted by rain	....	15 to 50
Sycamore, perfectly dry (see foot-note)	.59	37
Shales, red or black, 2.4 to 2.8	2.6	162
Silver	10.5	655
Slate, 2.7 to 2.9	2.8	175
Soapstone, 2.65 to 2.8	2.73	170
Spruce, perfectly dry (see foot-note)	.4	25
Steel	7.85	490
Sulphur	2.00	125
Tallow	.94	58.6
Tar	1	62.355
Tin, cast, 7.2 to 7.5	7.35	459
Walnut, black, perfectly dry (see foot-note)	.61	38
Water, pure rain, distilled, at 32° F., bar. 30 ins.	....	62.417
" " " " at 62° F., bar. 30 ins.	1	62.355
" " " " at 212° F., bar. 30 ins.	....	59.7
" sea, 1.026 to 1.030	1.028	64.08
Zinc or spelter, 6.8 to 7.2	7.00	437.5

Note.—Green timbers usually weigh from one fifth to nearly one half more than dry; ordinary building timbers, tolerably seasoned, one sixth more.

Specific Gravity.

The specific gravity of a substance is the number which expresses the ratio that the weight of a given volume of the substance bears to the weight of the same volume of distilled water at a temperature of 62° Fahr.; or, the specific gravity of a body is equal to its weight divided by the weight of an equal volume of water. The specific gravity of a substance, multiplied by the weight of a cubic foot of water, will give the weight of a cubic foot of the given substance.

The weight of a cubic foot of water, at 62° Fahr. and at the sea-level, is about 62.355 lbs.*

The specific gravity of a solid substance may be determined by first weighing a portion of it in air and then in water and dividing the weight in air by the loss of the weight in water; the quotient is the specific gravity required.

Example.—A piece of granite weighs 5.32 lbs. in air; when immersed in water it weighs 3.32 lbs.

Weight in air (5.32 lbs. divided by loss of weight in water (2 lbs.) = 2.66, the specific gravity.

2.66×62.355 lbs. = 165.84 lbs. = weight per cubic foot.

Wire Gauges.

A "wire gauge" is a method of designating the diameter of wires or the thickness of sheets of metal by the numbers of a table arranged on a certain fixed basis. There are now nine or ten different gauges, resulting in great confusion. The table on the following page gives the diameter of the gauges in common use. The only legal gauge in this country is the U. S. standard gauge, described on p. 1438. It is used by most of the manufacturers of sheet iron and steel and tin-plate. The Brown & Sharpe gauge is commonly used for designating size of copper wires (see p. 1320); also for sheet copper and brass.

The American Steel and Wire Co. uses the old Washburn & Moen gauge for all their steel and iron wire and also for wire nails. The sectional areas for this gauge are given on p. 1349.

When placing orders for sheets and wire, it is always best to specify the weight per square or lineal foot or the thickness or diameter in thousandths of an inch.

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* The text-books differ slightly in regard to this value.

WIRE AND SHEET-METAL GAUGES COMPARED.

(In decimals of an inch.)

Number of Gauge.	Birmingham or Stubs Iron Wire Gauge.	American or Brown & Sharpe Wire Gauge.	U. S. Standard Gauge for Sheet and Plate Iron and Steel.	Washburn & Moen Mfg. Co., John A. Roebling's Sons Co., and A. S. & W. Co. Wire Gauge.	Trenton Iron Co. Wire Gauge.	American Screw Co. Screw Wire Gauge.	British Imperial or English Legal Standard Wire Gauge.
7–0	....	....	.5	....	....	....	.500
6–0	....	....	.46875	.4600	....	....	.464
5–0	....	....	.4375	.4300	.450	....	.432
4–0	.454	.460000	.40625	.3938	.400	....	.400
3–0	.425	.409642	.375	.3625	.360	.0315	372
2–0	.380	.364796	.34375	.3310	.330	.0447	.348
0	.340	.324861	.3125	.3065	.305	.0578	.324
1	.300	.289297	.28125	.2830	.285	.0710	.300
2	.284	.257627	.265625	.2625	.265	.0842	.276
3	.259	.229423	.25	.2437	.245	.0973	.252
4	.238	.204307	.234375	.2253	.225	.1105	.232
5	.220	.181940	.21875	.2070	.205	.1236	.212
6	.203	.162023	.203125	.1920	.190	.1368	.192
7	.180	.144285	.1875	.1770	.175	.1500	.176
8	.165	.128490	.171875	.1620	.160	.1631	.160
9	.148	.114423	.15625	.1483	.145	.1763	.144
10	.134	.101897	.140625	.1350	.130	.1894	.128
11	.120	.090742	.125	.1205	.1175	.2026	.116
12	.109	.080808	.109375	.1055	.105	.2158	.104
13	.095	.071962	.09375	.0915	.0925	.2289	.092
14	.083	.064084	.078125	.0800	.0806	.2421	.080
15	.072	.057068	.0703125	.0720	.070	.2552	.072
16	.065	.050821	.0625	.0625	.061	.2684	.064
17	.058	.045257	.05625	.0540	.0525	.2816	.056
18	.049	.040303	.05	.0475	.045	.2947	.048
19	.042	.035890	.04375	.0410	.040	.3079	.040
20	.035	.031961	.0375	.0348	.035	.3210	.036
21	.032	.028462	.034375	.03175	.031	.3342	.032
22	.028	.025346	.03125	.0286	.028	.3474	.028
23	.025	.022572	.028125	.0258	.025	.3605	.024
24	.022	.020101	.025	.0230	.0225	.3737	.022
25	.020	.017900	.021875	.0204	.020	.3868	.020
26	.018	.015941	.01875	.0181	.018	.4000	.018
27	.016	.014195	.0171875	.0173	.017	.4132	.0164
28	.014	.012641	.015625	.0162	.016	.4263	.0148
29	.013	.011257	.0140625	.0150	.015	.4395	.0136
30	.012	.010025	.0125	.0140	.014	.4526	.0124
31	.010	.008928	.0109375	.0132	.013	.4658	.0116
32	.009	.007950	.01015625	.0128	.012	.4790	.0108
33	.008	.007080	.009375	.0118	.011	.4921	.0100
34	.007	.006305	.00859375	.0104	.010	.5053	.0092
35	.005	.005615	.0078125	.0095	.0095	.5184	.0084
36	.004	.005000	.00703125	.0090	.009	.5316	.0076
37	....	.004453	.006640625	.0085	.0085	.5448	.0068
38	....	.003965	.00625	.0080	.008	.5579	.0060
39	....	.003531	....	.0075	.0075	.5711	.0052
40	....	.003144	....	.0070	.007	.5842	.0048

WEIGHT PER SQUARE FOOT OF SHEETS OF WROUGHT IRON, STEEL, COPPER, AND BRASS.

(Thickness by American (B. & S.) Gauge.)

No. of Gauge.	Thickness in Inches.	Iron.	Steel.	Copper.	Brass.
0000	.46	18.46	18.70	20.84	19.69
000	.4096	16.44	16.66	18.56	17.53
00	.3648	14.64	14.83	16.53	15.61
0	.3249	13.04	13.21	14.72	13.90
1	.2893	11.61	11.76	13.11	12.38
2	.2576	10.34	10.48	11.67	11.03
3	.2294	9.21	9.33	10.39	9.82
4	.2043	8.20	8.31	9.26	8.74
5	.1819	7.30	7.40	8.24	7.79
6	.1620	6.50	6.59	7.34	6.93
7	.1443	5.79	5.87	6.54	6.18
8	.1285	5.16	5.22	5.82	5.50
9	.1144	4.59	4.65	5.18	4.90
10	.1019	4.09	4.14	4.62	4.36
11	.0907	3.64	3.69	4.11	3.88
12	.0808	3.24	3.29	3.66	3.46
13	.0720	2.89	2.93	3.26	3.08
14	.0641	2.57	2.61	2.90	2.74
15	.0571	2.29	2.32	2.59	2.44
16	.0508	2.04	2.07	2.30	2.18
17	.0453	1.82	1.84	2.05	1.94
18	.0403	1.62	1.64	1.83	1.73
19	.0359	1.44	1.46	1.63	1.54
20	.0320	1.28	1.30	1.45	1.37
21	.0285	1.14	1.16	1.29	1.22
22	.0253	1.02	1.03	1.15	1.08
23	.0226	.906	.918	1.02	.966
24	.0201	.807	.817	.911	.860
25	.0179	.718	.728	.811	.766
26	.0159	.640	.648	.722	.682
27	.0142	.570	.577	.643	.608
28	.0126	.507	.514	.573	.541
29	.0113	.452	.458	.510	.482
30	.0100	.402	.408	.454	.429
31	.0089	.358	.363	.404	.382
32	.0080	.319	.323	.360	.340
33	.0071	.284	.288	.321	.303
34	.0063	.253	.256	.286	.270
35	.0056	.225	.228	.254	.240
Specific gravity.....		7.704	7.806	8.698	8.218
Weight, cubic feet...		481.25	487.75	543.6	513.6
Weight, cubic ins....		.2787	.2823	.3146	.2972

WEIGHT OF SHEETS AND BARS OF LEAD, COPPER, AND BRASS.

Thickness or Diam. in Inches.	Lead.			Copper.			Brass.			Thickness or Diam. in Inches.
	Sheets, per Square Foot.	Square Bars 1 Foot Long.	Round Bars 1 Foot Long.	Sheets, per Square Foot.	Square Bars 1 Foot Long.	Round Bars 1 Foot Long.	Sheets, per Square Foot.	Square Bars 1 Foot Long.	Round Bars 1 Foot Long.	
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	
1/32	1.86	0.005	0.004	1.44	0.004	0.003	1.36	0.004	0.003	1/32
1/16	3.72	0.019	0.015	2.89	0.015	0.012	2.71	0.014	0.011	1/16
3/32	5.58	0.044	0.034	4.33	0.034	0.027	4.06	0.032	0.025	3/32
1/8	7.44	0.078	0.061	5.77	0.060	0.047	5.42	0.056	0.044	1/8
5/32	9.30	0.121	0.095	7.20	0.094	0.074	6.75	0.088	0.069	5/32
3/16	11.20	0.174	0.137	8.66	0.135	0.106	8.13	0.127	0.100	3/16
7/32	13.00	0.237	0.187	10.10	0.184	0.144	9.50	0.173	0.136	7/32
1/4	14.90	0.310	0.244	11.50	0.240	0.189	10.80	0.226	0.177	1/4
5/16	18.60	0.485	0.381	14.40	0.376	0.295	13.50	0.353	0.277	5/16
3/8	22.30	0.698	0.548	17.30	0.541	0.425	16.30	0.508	0.399	3/8
7/16	26.00	0.950	0.746	20.20	0.736	0.578	19.00	0.691	0.543	7/16
1/2	29.80	1.240	0.974	23.10	0.962	0.755	21.70	0.903	0.709	1/2
9/16	33.50	1.570	1.230	26.00	1.220	0.955	24.30	1.140	0.900	9/16
5/8	37.20	1.940	1.520	28.90	1.500	1.180	27.10	1.410	1.110	5/8
11/16	40.90	2.340	1.840	31.70	1.820	1.430	29.80	1.700	1.340	11/16
3/4	44.60	2.790	2.190	34.60	2.160	1.700	32.50	2.030	1.600	3/4
13/16	48.30	3.270	2.570	37.50	2.550	1.990	35.20	2.380	1.870	13/16
7/8	52.10	3.800	2.980	40.40	2.940	2.310	37.90	2.760	2.170	7/8
15/16	56.00	4.370	3.420	43.30	3.380	2.650	40.60	3.180	2.490	15/16
1	59.50	4.960	3.900	46.20	3.850	3.020	43.30	3.610	2.840	1
1 1/8	66.90	6.270	4.920	52.00	4.870	3.820	48.70	4.570	3.600	1 1/8
1 1/4	74.40	7.750	6.090	57.70	6.010	4.720	54.20	5.640	4.430	1 1/4
1 3/8	81.80	9.370	7.370	63.50	7.280	5.720	59.60	6.820	5.370	1 3/8
1 1/2	89.30	11.200	8.770	69.30	8.650	6.800	65.00	8.120	6.380	1 1/2
1 5/8	96.70	13.100	10.30	75.10	10.200	7.980	70.40	9.530	7.490	1 5/8
1 3/4	104.00	15.200	11.90	80.80	11.800	9.250	75.90	11.100	8.680	1 3/4
1 7/8	112.00	17.500	13.70	86.60	13.500	10.600	81.30	12.700	9.970	1 7/8
2	119.00	19.800	15.60	92.30	15.400	12.100	86.70	14.400	11.300	2

SIZE AND WEIGHT OF SMOOTH STEEL WIRE.

(As Made by the American Steel and Wire Company).

A. S. & W. Co. Gauge.	Diameter in Decimal of an Inch.	Sectional Area, Sq. Ins.	Approximate Weight of 100 Feet (Lbs.).	A. S. & W. Co. Gauge.	Diameter in Decimal of an Inch.	Sectional Area, Sq. Ins.	Approximate Weight of 100 Feet (Lbs.).
000	.3625	.1029	35.05	16	.0625	.00311	1.042
00	.3310	.0360	29.22	17	.0540	.00229	.7778
0	.3065	.0740	25.06	18	.0475	.00173	.6018
1	.2830	.0629	21.36	19	.0410	.00132	.4484
2	.2625	.0543	18.38	20	.0348	.00096	.3230
3	.2437	.0467	15.84	21	.0317	.00080	.2680
4	.2253	.0398	13.54	22	.0286	.00061	.2182
5	.2070	.0336	11.43	23	.0258	.00049	.1775
6	.1920	.0289	9.832	24	.0230	.00041	.1411
7	.1770	.0246	8.356	25	.0204	.00031	.1110
8	.1620	.0206	7.000	26	.0181	.00025	.08738
9	.1483	.0172	5.866	27	.0173	.00022	.07983
10	.1350	.0143	4.861	28	.0162	.00020	.07
11	.1205	.0113	3.873	29	.0150	.00017	.06001
12	.1055	.0086	2.969	30	.0140	.00015	.05228
13	.0915	.0066	2.233	31	.0132	.00014	.04647
14	.0800	.0050	1.707	32	.0128	.00013	.04370
15	.0720	.0041	1.383	33	.0118	.00009	.03714

Kinds of Wire Manufactured by the American Steel and Wire Company.

Market wire, Nos. 40 to 18.

Annealed stone or weaving wire, Nos. 16 to 47.

Tinned wire, Nos. 0 to 18.

Tinned stone wire, Nos. 18 to 36.

Gun screw wire, finished with great care as regards roundness and exactness to gauge, Nos. 50 to 18.

Machinery wire, Nos. 00000 to 18.

Cast-steel wire, ½-inch diameter, down to No. 20.

Drill and needle steel wire, Nos. 12 to 25.

The term "market wire" applies to the ordinary and most used forms of Bessemer *annealed, bright, galvanized, tinned,* and *coppered* wires.

Sectional area, weight, and strength of iron wire measured by the **Trenton Iron Company's** gauge is given on page 351.

WEIGHTS AND AREAS OF SQUARE AND ROUND BARS AND CIRCUMFERENCES OF ROUND BARS.

(Weights are for steel, at 489.6 lbs. per cu. ft.)

Thickness or Diameter in Inches.	Weight of □ Bar 1 Foot Long.	Weight of ○ Bar 1 Foot Long.	Area of □ Bar in Square Inches.	Area of ○ Bar in Square Inches.	Circumference of ○ Bar in Inches.
1/16	.013	.010	.0039	.0031	.1963
5/64	.021	.016	.0061	.0048	.2454
3/32	.030	.023	.0088	.0069	.2945
7/64	.041	.032	.0120	.0094	.3436
1/8	.053	.042	.0156	.0123	.3927
9/64	.067	.053	.0198	.0155	.4418
5/32	.083	.065	.0244	.0192	.4909
11/64	.100	.079	.0295	.0232	.5400
3/16	.120	.094	.0352	.0276	.5890
13/64	.140	.110	.0413	.0324	.6381
7/32	.163	.128	.0479	.0376	.6872
15/64	.187	.147	.0549	.0431	.7363
1/4	.213	.167	.0625	.0491	.7854
17/64	.240	.188	.0706	.0554	.8345
9/32	.269	.211	.0791	.0621	.8836
19/64	.300	.235	.0881	.0692	.9327
5/16	.332	.261	.0977	.0767	.9817
11/32	.402	.316	.1182	.0928	1.0799
3/8	.478	.376	.1406	.1104	1.1781
13/32	.561	.441	.1650	.1296	1.2763
7/16	.651	.511	.1914	.1503	1.3744
15/32	.747	.587	.2197	.1726	1.4726
1/2	.850	.668	.2500	.1963	1.5708
17/32	.960	.754	.2822	.2217	1.6690
9/16	1.076	.845	.3164	.2485	1.7671
19/32	1.199	.941	.3525	.2769	1.8653
5/8	1.328	1.043	.3906	.3068	1.9635
11/16	1.607	1.262	.4727	.3712	2.1598
3/4	1.913	1.502	.5625	.4418	2.3562
13/16	2.245	1.763	.6602	.5185	2.5525
7/8	2.603	2.044	.7656	.6013	2.74 9
15/16	2.989	2.347	.8789	.6903	2.9452

WEIGHTS AND AREAS OF SQUARE AND ROUND STEEL BARS.

(Weights are for steel, at 489.6 lbs. per cu. ft.)

Thickness, Ins.	□ Area.	□ Weight per Foot.	○ Area.	○ Weight per Foot.	Thickness, Ins.	□ Area.	□ Weight per Foot.	○ Area.	○ Wgt. per Foot.
1	1.000	3.400	.785	2.670	3	9.000	30.60	7.069	24.03
1/16	1.129	3.838	.887	3.014	1/16	9.379	31.89	7.366	25.04
1/8	1.266	4.303	.994	3.379	1/8	9.766	33.20	7.670	26.08
3/16	1.410	4.795	1.108	3.766	3/16	10.16	34.55	7.980	27.13
1/4	1.563	5.312	1.227	4.173	1/4	10.56	35.92	8.296	28.20
5/16	1.723	5.857	1.353	4.600	5/16	10.97	37.31	8.618	29.30
3/8	1.891	6.428	1.485	5.049	3/8	11.39	38.73	8.946	30.42
7/16	2.066	7.026	1.623	5.518	7/16	11.82	40.18	9.281	31.56
1/2	2.250	7.650	1.767	6.008	1/2	12.25	41.65	9.621	32.71
9/16	2.441	8.301	1.918	6.520	9/16	12.69	43.14	9.968	33.90
5/8	2.641	8.978	2.074	7.051	5/8	13.14	44.68	10.32	35.09
11/16	2.848	9.682	2.237	7.604	11/16	13.60	46.24	10.68	36.31
3/4	3.063	10.41	2.405	8.178	3/4	14.06	47.82	11.05	37.56
13/16	3.285	11.17	2.580	8.773	13/16	14.54	49.42	11.42	38.81
7/8	3.516	11.95	2.761	9.388	7/8	15.02	51.05	11.79	40.10
15/16	3.754	12.76	2.948	10.02	15/16	15.50	52.71	12.18	41.40
2	4.000	13.60	3.142	10.68	4	16.00	54.40	12.57	42.73
1/16	4.254	14.46	3.341	11.36	1/16	16.50	56.11	12.96	44.07
1/8	4.516	15.35	3.547	12.06	1/8	17.02	57.85	13.36	45.44
3/16	4.785	16.27	3.758	12.78	3/16	17.54	59.62	13.77	46.83
1/4	5.063	17.22	3.976	13.52	1/4	18.06	61.41	14.19	48.24
5/16	5.348	18.19	4.200	14.28	5/16	18.60	63.23	14.61	49.66
3/8	5.641	19.18	4.430	15.07	3/8	19.14	65.08	15.03	51.11
7/16	5.941	20.20	4.666	15.86	7/16	19.69	66.95	15.47	52.58
1/2	6.250	21.25	4.909	16.69	1/2	20.25	68.85	15.90	54.07
9/16	6.566	22.33	5.157	17.53	9/16	20.82	70.78	16.35	55.59
5/8	6.891	23.43	5.412	18.40	5/8	21.39	72.73	16.80	57.12
11/16	7.223	24.56	5.673	19.29	11/16	21.97	74.70	17.26	58.67
3/4	7.563	25.71	5.940	20.20	3/4	22.56	76.71	17.72	60.25
13/16	7.910	26.90	6.213	21.12	13/16	23.16	78.74	18.19	61.84
7/8	8.266	28.10	6.492	22.07	7/8	23.77	80.81	18.67	63.46
15/16	8.629	29.34	6.777	23.04	15/16	24.38	82.89	19.15	65.10

WEIGHTS AND AREAS OF SQUARE AND ROUND STEEL BARS—*Continued.*

(Weights are for steel, at 489.6 lbs. per cu. ft.)

Thickness, Ins.	□		○		Thickness, Ins.	□		○	
	Area.	Weight per Foot.	Area.	Weight per Foot		Area.	Weight per Foot.	Area.	Wgt. per Foot.
5	25.00	85.00	19.64	66.76	7	49.00	166.6	38.49	130.9
1/16	25.63	87.14	20.13	68.44	1/4	52.56	178.7	41.28	140.4
1/8	26.27	89.30	20.63	70.14	1/2	56.25	191.3	44.18	150.2
3/16	26.91	91.49	21.14	71.86	3/4	60.06	204.2	47.17	160.3
1/4	27.56	93.72	21.65	73.60	8	64.00	217.6	50.27	171.0
5/16	28.22	95.96	22.17	75.37	1/4	68.06	231.4	53.46	181.8
3/8	28.89	98.23	22.69	77.15	1/2	72.25	245.6	56.75	193.0
7/16	29.57	100.5	23.22	78.95	3/4	76.56	260.3	60.13	204.4
1/2	30.25	102.8	23.76	80.77	9	81.00	275.4	63.62	216.3
9/16	30.94	105.2	24.30	82.62	1/4	85.56	290.9	67.20	228.5
5/8	31.64	107.6	24.85	84.49	1/2	90.25	306.8	70.88	241.0
11/16	32.35	110.0	25.41	86.38	3/4	95.06	323.2	74.66	253.9
3/4	33.06	112.4	25.97	88.29	10	100.0	340.0	78.54	267.0
13/16	33.79	114.9	26.54	90.22	1/4	105.1	357.2	82.52	280.6
7/8	34.52	117.4	27.11	92.17	1/2	110.3	374.9	86.59	294.4
15/16	35.25	119.9	27.69	94.14	3/4	115.6	392.9	90.76	308.6
6	36.00	122.4	28.27	96.14	11	121.0	411.4	95.03	323.1
1/8	37.52	127.6	29.47	100.2	1/4	126.6	430.3	99.40	337.9
1/4	39.06	132.8	30.68	104.3	1/2	132.3	449.6	103.9	353.1
3/8	40.64	138.2	31.92	108.5	3/4	138.1	469.4	108.4	368.6
1/2	42.25	143.6	33.18	112.8	12	144.0	489.6	113.1	384.5
5/8	43.89	149.2	34.47	117.2					
3/4	45.56	154.9	35.79	121.7					
7/8	47.27	160.8	37.12	126.2					

Stock sizes of round and square bars vary by thirty seconds of an inch from 3/16 in. to 9/16 in. diameter, by sixteenths from 5/8 to 2 ins. diameter, by eighths from 2 to 3 ins. diameter, and by quarters of an inch from 3 ins. diameter and upwards. Round bars are also rolled by a few companies in sixty-fourths of an inch up to 1 in. diameter. Below 3/16 in. rounds are commonly designated by wire-gauge numbers.

WEIGHTS OF FLAT ROLLED STEEL BARS.

PER LINEAL FOOT.

(One cubic foot of steel weighs 489.6 lbs.)

For thicknesses from 1/16 inch to 9/16 inch and widths from 1/4 inch to 3/4 inch.

Thickness, in Inches.	Width in Inches.								
	1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″
1/16	.053	.066	.080	.093	.106	.120	.133	.146	.159
5/64	.066	.083	.100	.116	.133	.149	.166	.183	.199
3/32	.080	.100	.120	.139	.159	.179	.199	.219	.239
7/64	.093	.116	.139	.163	.186	.209	.232	.256	.279
1/8	.106	.133	.159	.186	.212	.239	.266	.292	.319
9/64	.120	.149	.179	.209	.239	.269	.299	.329	.359
5/32	.133	.166	.199	.232	.266	.299	.332	.365	.398
11/64	.146	.183	.219	.256	.292	.329	.365	.402	.438
3/16	.159	.199	.239	.279	.319	.359	.398	.438	.478
13/64	.173	.216	.259	.302	.345	.388	.432	.475	.518
7/32	.186	.232	.279	.325	.372	.418	.465	.511	.558
15/64	.199	.249	.299	.349	.398	.448	.498	.548	.598
1/4	.213	.266	.319	.372	.425	.478	.531	.584	.638
17/64	.226	.282	.339	.395	.452	.508	.564	.621	.677
9/32	.239	.299	.359	.418	.478	.538	.598	.657	.717
19/64	.252	.315	.379	.442	.505	.568	.631	.694	.757
5/16	.266	.332	.398	.465	.531	.598	.664	.730	.797
21/64	.279	.349	.418	.488	.558	.628	.697	.767	.827
11/32	.292	.365	.438	.511	.584	.657	.730	.804	.877
23/64	.305	.382	.458	.535	.611	.687	.764	.840	.916
3/8	.319	.398	.478	.558	.638	.717	.797	.877	.956
25/64	.332	.415	.498	.581	.664	.747	.830	.913	.996
13/32	.345	.432	.518	.604	.691	.777	.863	.950	1.04
27/64	.359	.448	.538	.628	.717	.807	.896	.986	1.03
7/16	.372	.465	.558	.651	.744	.837	.930	1.02	1.12
29/64	.385	.481	.578	.674	.770	.867	.963	1.06	1.16
15/32	.398	.498	.598	.697	.797	.896	.996	1.10	1.20
31/64	.412	.515	.618	.721	.823	.926	1.03	1.13	1.24
1/2	.425	.531	.638	.744	.850	.956	1.06	1.17	1.28
33/64	.438	.548	.657	.767	.877	.986	1.10	1.21	1.31
17/32	.452	.564	.677	.790	.903	1.02	1.13	1.24	1.35
35/64	.465	.581	.697	.813	.930	1.05	1.16	1.28	1.39
9/16	.478	.598	.717	.837	.956	1.08	1.20	1.31	1.43

WEIGHTS OF FLAT ROLLED STEEL BARS.—*Continued.*

PER LINEAL FOOT.

(1/16″ to 2″ in thickness, 1″ to 12″ in width.)

Thickness, in Inches.	1″	1¼″	1½″	1¾″	2″	2¼″	2½″	2¾″	3″
1/16	.21	.26	.32	.37	.43	.48	.53	.58	.63
1/8	.42	.53	.64	.75	.85	.96	1.06	1.17	1.28
3/16	.63	.79	.96	1.11	1.28	1.44	1.59	1.75	1.91
1/4	.85	1.06	1.28	1.49	1.70	1.91	2.12	2.34	2.55
5/16	1.06	1.33	1.59	1.86	2.12	2 39	2.65	2.92	3.19
3/8	1.28	1.59	1.92	2.23	2.55	2.87	3.19	3.51	3.83
7/16	1.49	1.86	2.23	2.60	2.98	3.35	3.72	4.09	4.46
1/2	1.70	2.12	2.55	2.98	3.40	3.83	4.25	4.67	5.10
9/16	1.92	2.39	2.87	3.35	3.83	4.30	4.78	5.26	5.74
5/8	2.12	2.65	3.19	3.72	4.25	4.78	5.31	5.84	6.38
11/16	2.34	2.92	3.51	4.09	4.67	5.26	5.84	6.43	7.02
3/4	2.55	3.19	3.83	4.47	5.10	5.75	6.38	7.02	7.65
13/16	2.76	3.45	4.14	4.84	5.53	6.21	6.90	7.60	8.29
7/8	2.98	3.72	4.47	5.20	5.95	6.69	7.44	8.18	8.93
15/16	3.19	3.99	4.78	5.58	6.38	7.18	7.97	8.77	9.57
1	3.40	4.25	5.10	5.95	6.80	7.65	8.50	9.35	10.20
1 1/16	3.61	4.52	5.42	6.32	7.22	8.13	9.03	9.93	10.84
1 1/8	3.83	4.78	5.74	6.70	7.65	8.61	9.57	10.52	11.48
1 3/16	4.04	5.05	6.06	7.07	8.08	9.09	10.10	11.11	12.12
1 1/4	4.25	5.31	6:38	7.44	8.50	9.57	10.63	11.69	12.75
1 5/16	4.46	5.58	6.69	7.81	8.93	10.04	11.16	12.27	13.39
1 3/8	4.67	5.84	7.02	8.18	9.35	10.52	11.69	12.85	14.03
1 7/16	4.89	6.11	7.34	8.56	9.78	11.00	12.22	13:44	14.66
1 1/2	5.10	6.38	7.65	8.93	10.20	11.48	12.75	14.03	15.30
1 9/16	5.32	6.64	7.97	9.30	10.63	11.95	13.28	14.61	15.94
1 5/8	5.52	6.90	8.29	9.67	11.05	12.43	13.81	15.19	16.58
1 11/16	5.74	7.17	8.61	10.04	11.47	12.91	14.34	15.78	17.22
1 3/4	5.95	7.44	8.93	10.42	11.90	13.40	14.88	16.37	17.85
1 13/16	6.16	7.70	9.24	10.79	12.33	13.86	15.40	16.95	18.49
1 7/8	6.38	7.97	9.57	11.15	12.75	14.34	15.94	17.53	19.13
1 15/16	6.59	8.24	9.88	11.53	13.18	14.83	16.47	18.12	19.77
2	6.80	8.50	10.20	11.90	13.60	15.30	17.00	18.70	20.40

WEIGHTS OF FLAT ROLLED STEEL BARS.—*Continued.*

PER LINEAL FOOT.

(1/16″ to 2″ in thickness, 1″ to 12″ in width.)

Thickness, in Inches.	3½″	4″	4½″	5″	5½″	6″	6½″	7″	7½″
1/16	.75	.85	.96	1.06	1.17	1.28	1.39	1.49	1.60
⅛	1.49	1.70	1.92	2.13	2.34	2.55	2.77	2.98	3.19
3/16	2.23	2.55	2.87	3.19	3.51	3.83	4.14	4.46	4.78
¼	2.98	3.40	3.83	4.25	4.67	5.10	5.53	5.95	6.36
5/16	3.72	4.25	4.78	5.31	5.84	6.38	6.90	7.44	7.97
⅜	4.47	5.10	5.74	6.38	7.02	7.65	8.29	8.93	9.57
7/16	5.20	5.95	6.70	7.44	8.18	8.93	9.67	10.41	11.16
½	5.95	6.80	7.65	8.50	9.35	10.20	11.05	11.90	12.75
9/16	6.70	7.65	8.61	9.57	10.52	11.48	12.43	13.39	14.34
⅝	7.44	8.50	9.57	10.63	11.69	12.75	13.81	14.87	15.94
11/16	8.18	9.35	10.52	11.69	12.85	14.03	15.20	16.36	17.53
¾	8.93	10.20	11.48	12.75	14.03	15.30	16.58	17.85	19.13
13/16	9.67	11.05	12.43	13.81	15.19	16.58	17.95	19.34	20.72
⅞	10.41	11.90	13.39	14.87	16.36	17.85	19.34	20.83	22.32
15/16	11.16	12.75	14.34	15.94	17.53	19.13	20.72	22.32	23.91
1	11.90	13.60	15.30	17.00	18.70	20.40	22.10	23.80	25.50
1 1/16	12.65	14.45	16.26	18.06	19.87	21.68	23.48	25.29	27.10
1⅛	13.39	15.30	17.22	19.13	21.04	22.95	24.87	26.78	28.68
1 3/16	14.13	16.15	18.17	20.19	22.21	24.23	26.24	28.26	30.28
1¼	14.87	17.00	19.13	21.25	23.38	25.50	27.62	29.75	31.88
1 5/16	15.62	17.85	20.08	22.32	24.54	26.78	29.01	31.23	33.48
1⅜	16.36	18.70	21.04	23.38	25.71	28.05	30.39	32.72	35.06
1 7/16	17.10	19.85	21.99	24.44	26.88	29.33	31.77	34.21	36.66
1½	17.85	20.40	22.95	25.50	28.05	30.60	33.15	35.70	38.26
1 9/16	18.60	21.25	23.91	26.57	29.22	31.88	34.53	37.19	39.84
1⅝	19.34	22.10	24.87	27.63	30.39	33.15	35.91	38.67	41.44
1 11/16	20.08	22.95	25.82	28.69	31.55	34.43	37.30	40.16	43.03
1¾	20.83	23.80	26.78	29.75	32.73	35.70	38.68	41.65	44.63
1 13/16	21.57	24.65	27.73	30.81	33.89	36.98	40.05	43.14	46.22
1⅞	22.31	25.50	28.69	31.87	35.06	38.25	41.44	44.63	47.82
1 15/16	23.06	26.35	29.64	32.94	36.23	39.53	42.82	46.12	49.41
2	23.80	27.20	30.60	34.00	37.40	40.80	44.20	47.60	51.00

WEIGHTS OF FLAT ROLLED STEEL BARS.—*Continued.*

PER LINEAL FOOT.

(1/16″ to 2″ in thickness, 1″ to 12″ in width.)

Thickness, in Inches.	8″	8½″	9″	9½″	10″	10½″	11″	11½″	12″
1/16	1.70	1.81	1.91	2.02	2.13	2.23	2.34	2.45	2.55
⅛	3.40	3.61	3.82	4.04	4.25	4.46	4.68	4.89	5.10
3/16	5.10	5.42	5.74	6.06	6.38	6.70	7.02	7.32	7.65
¼	6.80	7.22	7.65	8.08	8.50	8.92	9.34	9.78	10.20
5/16	8.50	9.03	9.56	10.10	10.62	11.16	11.68	12.22	12.75
⅜	10.20	10.84	11.48	12.12	12.75	13.39	14.03	14.68	15.30
7/16	11.90	12.64	13.40	14.14	14.88	15.62	16.36	17.12	17.85
½	13.60	14.44	15.30	16.16	17.00	17.85	18.70	19.55	20.40
9/16	15.30	16.26	17.22	18.18	19.14	20.08	21.02	22.00	22.95
⅝	17.00	18.06	19.13	20.19	21.25	22.32	23.38	24.44	25.50
11/16	18.70	19.86	21.04	22.21	23.38	24.54	25.70	26.88	28.05
¾	20.40	21.68	22.96	24.23	25.50	26.78	28.05	29.33	30.60
13/16	22.10	23.48	24.86	26.24	27.62	29.00	30.40	31.76	33.15
⅞	23.80	25.30	26.78	28.26	29.75	31.24	32.72	34.21	35.70
15/16	25.50	27.10	28.69	30.28	31.88	33.48	35.06	36.66	38.25
1	27.20	28.90	30.60	32.30	34.00	35.70	37.40	39.10	40.80
1 1/16	28.90	30.70	32.52	34.32	36.12	37.92	39.74	41.54	43.35
1⅛	30.60	32.52	34.43	36.34	38.25	40.17	42.08	44.00	45.90
1 3/16	32.30	34.32	36.34	38.36	40.38	42.40	44.42	46.44	48.45
1¼	34.00	36.12	38.26	40.37	42.50	44.63	46.76	48.88	51.00
1 5/16	35.70	37.93	40.16	42.40	44.64	46.86	49.08	51.32	53.55
1⅜	37.40	39.74	42.08	44.41	46.75	49.08	51.42	53.76	56.10
1 7/16	39.10	41.54	44.00	46.44	48.88	51.32	53.76	56.21	58.65
1½	40.80	43.35	45.90	48.45	51.00	53.55	56.10	58.65	61.20
1 9/16	42.50	45.16	47.82	50.48	53.14	55.78	58.42	61.10	63.75
1⅝	44.20	46.96	49.73	52.49	55.25	58.02	60.78	63.54	66.30
1 11/16	45.90	48.76	51.64	54.51	57.38	60.24	63.10	65.98	68.85
1¾	47.60	50.58	53.56	56.53	59.50	62.48	65.45	68.43	71.40
1 13/16	49.30	52.38	55.46	58.54	61.62	64.70	67.80	70.86	73.95
1⅞	51.00	54.20	57.38	60.56	63.75	66.94	70.12	73.31	76.50
1 15/16	52.70	56.00	59.29	62.58	65.88	69.18	72.46	75.76	79.05
2	54.40	57.80	61.20	64.60	68.00	71.40	74.80	78.20	81.60

Rules for Estimating the Weight of any Piece of Wrought Iron, Steel, or Cast Iron.

Wrought iron:

One cubic foot of wrought iron weighs... 480 lbs.
One square foot, one inch thick, weighs.. 40 "
One square inch, one foot long, weighs... $3\frac{1}{3}$ "

To find the weight per square foot of sheet iron, multiply the thickness in inches by 40.

To find the weight per lineal foot of bars of any section, multiply the cross-sectional area in square inches by $3\frac{1}{3}$.

Steel:

One cubic foot of steel weighs......... 489.6 lbs.
(Or just 2 per cent. more than wrought iron.)
One square foot, one inch thick, weighs.. 40.8 "
One square inch, one foot long, weighs.. 3.4 "

To find the weight per lineal foot, of bars of any section, multiply the cross-sectional area in square inches by 3.4; or, if the weight is known, the *exact* sectional area may be obtained by dividing by 3.4.

Cast iron:

One cubic foot of cast iron weighs....... 450 lbs.
One square foot, one inch thick, weighs.. $37\frac{1}{2}$ "
One square inch, one foot long, weighs... $3\frac{1}{8}$ "
One cubic inch weighs.................... .26 "

The weight of irregular castings must be estimated by the cubic inch.

Rules for Weights of Castings.

Multiply the weight of the pattern by 18 for cast iron, 13 for brass, 19 for lead, 12.2 for tin, 11.4 for zinc, and the product is the weight of the casting.

Reduction for Round Cores and Core Prints.

RULE.—Multiply the square of the diameter by the length of the core in inches, and the product multiplied by 0.017 is the weight of the pine core to be deducted from the weight of the pattern.

Shrinkage in Castings.

Pattern-makers' Rule. { Cast iron, $\frac{1}{8}$; Brass .... $\frac{3}{16}$; Lead. .... $\frac{1}{8}$; Tin. .....$\frac{1}{12}$; Zinc .... .$\frac{3}{16}$ } of an inch longer per lineal foot.

WEIGHT OF SQUARE CAST-IRON COLUMNS IN POUNDS PER LINEAL FOOT.

(Birkmire.)

$a\square b$	Thickness of Metal in Inches.							
$2a+2b$	⅝	¾	⅞	1	1⅛	1¼	1½	1¾	2
*									
12	18.6	21.1	23.3	25.0	26.4	27.3	28.1		
14	22.5	25.8	28.7	31.3	33.4	35.1	37.5		
16	26.4	30.5	34.2	37.5	40.4	43.0	46.9	49.2	50.0
18	30.3	35.2	39.7	43.8	47.4	50.8	56.3	60.2	62.5
20	34.2	39.8	45.1	50.0	54.5	58.6	65.6	71.1	75.0
22	38.1	44.5	50.6	56.3	61.5	66.4	75.0	82.0	87.5
24	42.0	49.2	56.1	62.5	68.5	74.2	84.4	93.0	100.0
26	45.9	53.9	61.5	68.8	75.6	82.0	93.8	103.9	112.5
28	49.8	58.6	67.0	75.0	82.6	89.8	103.1	114.8	125.0
30	53.7	63.3	72.5	81.3	89.6	97.7	112.5	125.8	137.5
32	57.6	68.0	77.9	87.5	96.7	105.5	121.9	136.7	150.0
34	61.5	72.7	83.4	93.8	103.7	113.3	131.3	147.7	162.5
36	65.4	77.3	88.9	100.0	110.7	121.1	140.6	158.6	175.0
38	69.3	82.0	94.3	106.3	117.8	128.9	150.0	169.5	187.5
40	73.2	86.7	99.8	112.5	124.8	136.7	159.4	180.5	200.0
42	77.1	91.4	105.3	118.8	131.8	144.5	168.8	191.4	212.5
44	81.0	96.1	110.8	125.0	138.8	152.3	178.1	202.3	225.0
46	84.9	100.8	116.2	131.3	145.9	160.2	187.5	213.3	237.5
48	88.8	105.5	121.7	137.5	152.9	168.0	196.9	224.2	250.0
50	92.8	110.2	127.2	143.8	159.9	175.8	206.3	235.2	262.5
52	96.7	114.8	132.6	150.0	167.0	183.6	215.6	246.1	275.0
54	100.6	11[illegible].5	138.1	156.3	174.0	191.4	225.0	257.0	287.5
56	104.5	124.2	143.6	162.5	181.0	199.2	234.4	268.0	300.0
58	108.4	128.9	149.0	168.8	188.1	207.0	243.8	278.9	312.5
60	112.3	133.6	154.5	175.0	195.1	214.9	253.2	289.8	325.0
62	116.2	138.3	160.0	181.3	202.1	222.7	262.5	300.8	337.5
64	120.1	143.0	165.4	187.5	209.2	230.5	271.9	311.7	350.0
66	124.0	147.7	170.9	193.8	216.2	238.3	281.3	322.7	362.5
68	127.9	152.3	176.4	200.0	223.2	246.1	290.6	333.6	375.0
70	131.8	157.0	181.8	206.3	230.3	253.9	300.0	344.5	387.5
72	135.7	161.7	187.3	212.5	237.3	261.7	309.4	355.5	400.0
74	139.6	166.4	192.8	218.8	244.3	269.5	318.8	366.4	412.5
76	143.5	171.1	198.3	225.0	251.3	277.3	328.1	377.3	425.0
78	147.4	175.8	203.7	231.3	258.4	285.2	337.5	388.3	437.5
80	151.3	180.5	207.2	237.5	265.4	293.0	346.9	399.2	450.0

* a and b = either side (outside measurement). $2a+2b$ = number. Allowance has been made in this table for corners counted twice.

EXAMPLE.—What is the weight per lineal foot of a 12″×16″ ×1″ thick column?

Ans.—$2a+2b=24+32=56$. Opposite this number, under 1-inch thick metal, we find 162.5, or weight per lineal foot of a 12″×16″×1″ thick column.

NOTE.—For flanges, brackets, etc., calculate the cubical contents of same and multiply by .26; cast iron averaging 450 pounds per cubic foot.

WEIGHT PER LINEAL FOOT OF CIRCULAR CAST-IRON COLUMNS. (BIRKMIRE.)

Outside Diam., in Inches.	Thickness of Metal, in Inches.																
	½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½
3	12.3	14.6	16.6	18.3	19.6												
4	17.2	21.0	24.0	27.0	29.5	32.1	33.8	35.4									
5	22.1	27.0	31.3	35.5	39.3	43.0	46.0	49.0	51.54	54.1	55.84	57.5					
6	27.0	33.0	39.0	44.0	49.1	54.1	58.3	62.4	66.30	69.9	73.02	76.0	78.6	80.84	82.83		
7	32.0	39.1	46.0	53.0	59.0	65.1	70.6	76.1	81.00	85.6	90.20	94.3	98.2	101.70	105.00	107.84	110.45
8	36.8	45.3	53.4	61.2	69.1	76.1	83.1	89.5	95.80	101.8	107.40	112.8	117.8	122.60	127.00	131.20	135.00
9	41.7	51.4	61.1	70.0	78.6	87.1	95.1	103.1	110.50	117.7	124.60	131.2	137.5	143.40	149.10	154.50	159.50
10	46.6	57.5	68.13	78.41	88.4	98.0	107.4	116.4	125.20	133.7	142.00	149.6	157.1	164.30	171.20	177.80	184.10
11	51.6	64.0	75.5	87.1	98.2	109.1	120.1	130.1	140.00	149.6	159.00	168.0	176.8	185.20	193.30	201.10	208.60
12	56.5	70.0	82.87	96.1	108.0	120.0	132.1	143.5	154.70	165.6	176.00	186.4	196.4	206.00	215.40	224.40	233.20
13	61.4	76.0	90.23	104.2	118.1	131.2	144.2	157.1	169.40	181.5	193.30	204.8	216.0	226.90	237.50	247.70	257.70
14	66.3	82.1	97.6	113.2	128.1	142.0	156.5	170.4	184.10	197.4	210.50	223.2	235.7	247.70	259.60	271.10	282.30
15	71.2	88.2	104.96	121.4	137.5	153.3	169.4	184.1	198.90	213.4	227.70	241.6	255.3	268.20	281.70	294.40	306.80
16	76.1	94.4	112.33	130.1	147.3	164.3	181.0	197.4	213.50	229.4	244.90	260.0	274.9	289.50	303.70	317.70	331.50
17	81.0	100.5	120.1	139.1	157.1	175.4	193.3	211.0	228.30	245.3	262.00	278.4	294.5	310.30	325.80	341.00	355.90
18	86.0	107.0	127.0	147.0	167.0	186.4	206.0	224.4	243.00	261.3	279.20	296.8	314.2	331.20	348.00	364.30	380.40
19	91.0	113.0	134.4	156.0	177.1	197.5	218.1	238.0	257.70	277.2	296.40	315.2	338.8	352.10	370.00	387.70	405.00
20	96.0	119.0	142.1	164.3	186.6	208.8	230.1	251.5	272.50	293.2	313.60	333.6	353.4	372.90	392.10	411.00	430.00
21	100.6	125.0	149.1	173.1	196.6	219.6	242.4	265.0	287.20	309.0	330.80	352.1	373.1	393.80	414.20	434.30	454.10
22	105.6	131.2	156.5	181.5	206.2	230.6	255.0	278.0	302.00	325.1	348.00	370.5	393.0	414.60	436.30	457.60	478.60
23	110.5	137.3	164.1	190.1	216.1	242.0	267.0	292.0	316.70	341.0	365.10	388.9	412.3	435.50	458.40	481.00	503.20
24	115.4	143.5	171.2	199.0	226.0	253.0	279.2	305.4	331.40	357.0	382.30	407.3	432.0	456.40	480.50	504.20	527.70

Note.—The table is arranged for the weight of plain shaft. For brackets, flanges, etc., calculate the cubical contents and multiply by .26.

WEIGHT OF CAST-IRON PLATES.

WEIGHT, IN POUNDS, OF CAST-IRON PLATES ONE INCH THICK.

(Calculated at 450 lbs. per cubic foot.)

Length, in Ins	Width, in Inches.									
	6	8	10	12	14	16	18	20	24	30
4	6.25	8.3	10.4	12.5	14.6	16.6	18.7	20.8	25	31
6	9.37	12.5	15.6	18.7	21.8	25.0	28.1	31.2	38	47
8	12.50	16.6	20.8	25.0	29.1	33.3	37.4	41.6	50	62
10	15.60	20.8	26.0	31.2	36.4	41.6	46.8	52.0	63	78
12	18.70	25.0	31.2	37.5	43.7	49.9	56.2	62.4	75	94
14	21.80	29.2	36.4	43.7	51.0	58.2	65.5	72.8	88	109
16	24.90	33.3	41.6	50.0	58.2	66.6	74.9	83.2	100	125
18	28.10	37.5	46.8	56.2	65.5	74.9	84.2	93.6	113	140
20	31.20	41.6	52.0	62.3	72.8	83.2	93.6	104.0	125	156
22	34.30	45.8	57.2	68.6	80.1	91.5	103.0	114.4	138	172
24	37.50	50.0	62.4	75.0	87.4	99.8	112.3	124.8	150	187
26	40.60	54.0	67.6	81.2	94.6	108.2	121.7	135.2	163	203
28	43.60	58.2	72.8	87.5	101.9	116.5	131.0	145.6	175	218
30	46.80	62.4	78.0	93.7	109.2	124.8	140.4	156.0	188	234
32	49.80	66.6	83.2	100.0	116.5	133.1	150.3	166.4	200	250
36	56.10	75.0	93.6	112.5	131.0	150.0	168.4	187.2	225	281

For larger plates take size of plate *one half* smaller and multiply by 2. Thus a plate 28″×32″ will weigh twice as much as one 14″×32″. For plates more or less than one inch in thickness multiply weight of plate by thickness in inches.

APPROXIMATE WEIGHT OF SQUARE-RIBBED CAST-IRON COLUMN BASES.

The following table, giving the weight of cast-iron column bases, is new and will be useful when estimating the steel and iron in tall buildings: *

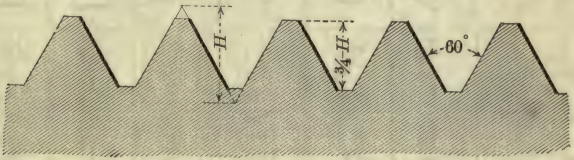
Size of Square Base.	Weight in Pounds.	Size of Square Base.	Weight in Pounds.
22×22	600	32×32	1,340
24×24	750	34×34	1,450
26×26	880	36×36	1,600
28×28	1,020	38×38	1,720
30×30	1,180	40×40	1,850

* H. G. Tyrrell, C.E., in *Architects and Builders Magazine*, Jan., 1903.

SCREW-THREADS, NUTS, AND BOLT-HEADS.

STANDARD SCREW-THREADS.

Recommended by Franklin Institute, Dec. 15, 1864, and adopted by Navy Dept of the United States: by the R. R. Master Mechanics' and Master Car-builders' Associations; by Messrs. Jones & Laughlins, Limited; and by many other of the prominent engineering and mechanical establishments of the country.



Angle of thread 60° Flat at top and bottom ⅛ of pitch.

Diam. of Screw.	Threads per Inch.	Diam. at Root of Thread.	Area at Root of Thread.	Diam. of Screw.	Threads per Inch.	Diam. at Root of Thread.	Area at Root of Thread.
			sq. in.				sq. in.
¼	20	.185	.027	2	4½	1.712	2.302
5/16	18	.240	.045	2¼	4½	1.962	3.023
⅜	16	.294	.068	2½	4	2.176	3.719
7/16	14	.344	.093	2¾	4	2.426	4.620
½	13	.400	.126	3	3⅝	2.629	5.428
9/16	12	.454	.162	3¼	3½	2.879	6.510
⅝	11	.507	.202	3½	3	3.100	7.548
¾	10	.620	.302	3¾	3	3.317	8.641
⅞	9	.731	.420	4	3	3.567	9.963
1	8	.837	.550	4¼	2	3.798	11.329
1⅛	7	.940	.694	4½	2	4.028	12.753
1¼	7	1.065	.893	4¾	2	4.256	14.226
1⅜	6	1.160	1.057	5	2	4.480	15.763
1½	6	1.284	1.295	5¼	2	4.730	17.572
1⅜	5½	1.389	1.515	5½	2	4.953	19.267
1¾	5	1.491	1.746	5¾	2	5.203	21.262
1⅞	5	1.616	2.051	6	2	5.423	23.098

Nuts and Bolt-heads are determined by the following rules, which apply to both square and hexagon nuts:

Short diameter of rough nut = 1½ × diam. of bolt + ⅛ in.

Short diameter of finished nut = 1½ × diam. of bolt + 1/16 in.

Thickness of rough nut = diam. of bolt.

Thickness of finished nut = diam. of bolt − 1/16 in.

Short diameter of rough head = 1½ × diam. of bolt + ⅛ in.

Short diameter of finished head = 1½ × diam. of bolt + 1/16 in.

Thickness of rough head = ½ short diam. of head.

Thickness of finished head = diam. of bolt − 1/16 in.

The long diameter of a hexagon nut may be obtained by multiplying the short diameter by 1.155, and the long diameter of a square nut by multiplying the short diameter by 1.414.

STANDARD DIMENSIONS OF NUTS AND BOLT-HEADS.

Dia. of Bolt.	Short Diam. Rough.	Short Diam. Finish.	Long Diam. Rough.	Long Diam. Rough.	Thickness, Rough, Nut.	Thickness, Finish. Both.	Thickness, Rough. Head.
1/4	1/2	7/16	37/64	7/10	1/4	3/16	1/4
5/16	19/32	17/32	11/16	10/12	5/16	1/4	19/64
3/8	11/16	5/8	51/64	63/64	3/8	5/16	11/32
7/16	25/32	23/32	9/10	1 7/64	7/16	3/8	25/64
1/2	7/8	13/16	1	1 15/64	1/2	7/16	7/16
9/16	31/32	29/32	1 1/8	1 23/64	9/16	1/2	31/64
5/8	1 1/16	1	1 7/32	1 1/2	5/8	9/16	17/32
3/4	1 1/4	1 3/16	1 7/16	1 49/64	3/4	11/16	5/8
7/8	1 7/16	1 3/8	1 21/32	2 1/32	7/8	13/16	23/32
1	1 5/8	1 9/16	1 7/8	2 19/64	1	15/16	13/16
1 1/8	1 13/16	1 3/4	2 3/32	2 9/16	1 1/8	1 1/16	29/32
1 1/4	2	1 15/16	2 5/16	2 53/64	1 1/4	1 3/16	1
1 3/8	2 3/16	2 1/8	2 17/32	3 3/32	1 3/8	1 5/16	1 3/32
1 1/2	2 3/8	2 5/16	2 3/4	3 23/64	1 1/2	1 7/16	1 3/16
1 5/8	2 9/16	2 1/2	2 12/32	3 5/8	1 5/8	1 9/16	1 9/32
1 3/4	2 3/4	2 11/16	3 3/16	3 57/64	1 3/4	1 11/16	1 3/8
1 7/8	2 15/16	2 7/8	3 13/32	4 5/32	1 7/8	1 13/16	1 15/32
2	3 1/8	3 1/16	3 5/8	4 27/64	2	2 15/16	1 9/16
2 1/4	3 1/2	3 7/16	4 1/16	4 61/64	2 1/4	2 3/16	1 3/4
2 1/2	3 7/8	3 13/16	4 1/2	5 31/64	2 1/2	2 7/16	1 15/16
2 3/4	4 1/4	4 3/16	4 29/32	6	2 3/4	2 11/16	2 1/8
3	4 5/8	4 9/16	5 3/8	6 17/32	3	2 15/16	2 5/16
3 1/4	5	4 15/16	5 13/16	7 1/16	3 1/4	3 3/16	2 1/2
3 1/2	5 3/8	5 5/16	6 7/64	7 39/64	3 1/2	3 7/16	2 11/16
3 3/4	5 3/4	5 11/16	6 21/32	8 1/8	3 3/4	3 13/16	2 7/8
4	6 1/8	6 1/16	7 3/32	8 41/64	4	3 15/16	3 1/16
4 1/4	6 1/2	6 7/16	7 9/16	9 3/16	4 1/4	4 3/16	3 1/4
4 1/2	6 7/8	6 13/16	7 31/32	9 3/4	4 1/2	4 7/16	3 7/16
4 3/4	7 1/4	7 3/16	8 13/32	10 1/4	4 3/4	4 11/16	3 5/8
5	7 5/8	7 9/16	8 27/32	10 49/64	5	4 15/16	3 13/16
5 1/4	8	7 15/16	9 9/32	11 23/64	5 1/4	5 3/16	4
5 1/2	8 3/8	8 5/16	9 23/32	11 7/8	5 1/2	5 7/16	4 3/16
5 3/4	8 3/4	8 11/16	10 5/32	12 3/8	5 3/4	5 11/16	4 3/8
6	9 1/8	9 1/16	10 19/32	12 15/16	6	5 15/16	4 9/16

WEIGHT OF ONE HUNDRED BOLTS WITH SQUARE HEADS AND NUTS.

INCLUDES WEIGHT OF NUT.

(Hoopes & Townsend's List.)

Length under Head to Point.	Diameter of Bolts, Inches.								
	¼	5/16	⅜	7/16	½	⅝	¾	⅞	1
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
1½	4.00	7.00	10.50	15.20	22.50	39.50	63.00	—	—
1¾	4.35	7.50	11.25	16.30	23.82	41.62	66.00	—	—
2	4.75	8.00	12.00	17.40	25.15	43.75	69.00	109.00	163
2¼	5.15	8.50	12.75	18.50	26.47	45.88	72.00	113.25	169
2½	5.50	9.00	13.50	19.60	27.80	48.00	75.00	117.50	174
2¾	5.75	9.50	14.25	20.70	29.12	50.12	78.00	121.75	180
3	6.25	10.00	15.00	21.80	30.45	52.25	81.00	126.00	185
3½	7.00	11.00	16.50	24.00	33.10	56.50	87.00	134.25	196
4	7.75	12.00	18.00	26.20	35.75	60.75	93.10	142.50	207
4½	8.50	13.00	19.50	28.40	38.40	65.00	99.05	151.00	218
5	9.25	14.00	21.00	30.60	41.05	69.25	105.20	159.55	229
5½	10.00	15.00	22.50	32.80	43.70	73.50	111.25	168.00	240
6	10.75	16.00	24.00	35.00	46.35	77.75	117.30	176.60	251
6½	—	—	25.50	37.20	49.00	82.00	123.35	185.00	262
7	—	—	27.00	39.40	51.65	86.25	129.40	193.65	273
7½	—	—	28.50	41.60	54.30	90.50	135.00	202.00	284
8	—	—	30.00	43.80	59.60	94.75	141.50	210.70	295
9	—	—	—	46.00	64.90	103.25	153.60	227.75	317
10	—	—	—	48.20	70.20	111.75	165.70	224.80	339
11	—	—	—	50.40	75.50	120.25	177.80	261.85	360
12	—	—	—	52.60	80.80	128.75	189.90	278.90	382
13	—	—	—	—	86.10	137.25	202.00	295.95	404
14	—	—	—	—	91.40	145.75	214.10	313.00	426
15	—	—	—	—	96.70	154.25	226.20	330.05	448
16	—	—	—	—	102.00	162.75	238.30	347.10	470
17	—	—	—	—	107.30	171.00	250.40	364.15	492
18	—	—	—	—	112.60	179.50	262.60	381.20	514
19	—	—	—	—	117.90	188.00	274.70	398.25	536
20	—	—	—	—	123.20	206.50	286.80	415.30	558
Per inch addit'l	1.37	2.13	3.07	4.18	5.45	8.52	12.27	16.70	21.82

WEIGHTS OF NUTS AND BOLT-HEADS, IN POUNDS.

(For calculating the weight of longer bolts.)

Diameter of Bolt, in Inches.		¼	⅜	½	⅝	¾	⅞
Weight of hexagon nut and head.	—	0.017	0.057	0.128	0.267	0.43	0.73
Weight of square nut and head.	—	0.021	0.069	0.164	0.320	0.55	0.88

Diameter of Bolt, in Inches.	1	1¼	1½	1¾	2	2½	3
Weight of hexagon nut and head.	1.10	2.14	3.78	5.6	8.75	17	28.8
Weight of square nut and head.	1.31	2.56	4.42	7.0	10.50	21	36.4

WEIGHT OF RIVETS AND ROUND-HEADED BOLTS WITHOUT NUTS—STEEL.

POUNDS PER HUNDRED.

Length, Inches.	3/8 In. Diam.	1/2 In. Diam.	5/8 In. Diam.	3/4 In. Diam.	7/8 In. Diam.	1 In. Diam.	1 1/8 In. Diam.	1 1/4 In. Diam.
1 1/4	5.5	12.8	22.0	29.3	43.9	66.6	93.3	127.
1 1/2	6.3	14.2	24.1	32.4	48.2	72.1	100.	136.
1 3/4	7.0	15.5	26.3	35.5	52.5	77.7	107.	145.
2	7.9	16.9	28.5	38.7	56.7	83.3	114.	153.
2 1/4	8.7	18.3	30.7	41.8	61.0	88.8	121.	162.
2 1/2	9.4	19.7	32.8	44.9	65.2	94.4	128.	171.
2 3/4	10.2	21.1	35.0	48.0	69.5	100.	136.	179.
3	11.0	22.5	37.2	51.1	73.7	105.	143.	188.
3 1/4	11.7	23.9	39.3	54.3	78.0	111.	150.	197.
3 1/2	12.6	25.3	41.5	57.4	82.3	116.	157.	205.
3 3/4	13.4	26.7	43.7	60.5	86.5	122.	164.	214.
4	14.1	28.1	45.9	63.6	90.8	128.	170.	223.
4 1/4	14.9	29.4	48.0	66.7	95.0	134.	177.	231.
4 1/2	15.7	30.8	50.2	69.9	99.3	139.	185.	240.
4 3/4	16.5	32.2	52.4	73.0	104.	145.	192.	249.
5	17.2	33.6	54.5	76.1	108.	150.	199.	258.
5 1/4	18.1	35.0	56.7	79.2	112.	156.	206.	266.
5 1/2	18.8	36.4	58.9	82.3	116.	161.	213.	275.
5 3/4	19.6	37.8	61.1	85.5	120.	166.	220.	284.
6	20.4	39.2	63.2	88.6	124.	172.	227.	292.
6 1/2	21.9	42.0	67.6	95.1	133.	184.	241.	310.
7	23.5	44.7	71.9	101.	142.	195.	255.	327.
7 1/2	25.1	47.5	76.1	108.	150.	206.	269.	345.
8	26.6	50.3	80.6	114.	159.	217.	284.	362.
8 1/2	28.2	53.1	85.0	120.	167.	227.	298.	379.
9	29.8	55.9	89.3	126.	176.	239.	312.	397.
9 1/2	31.3	58.7	93.7	133.	185.	250.	325.	414.
10	32.8	61.4	98.0	139.	193.	261.	340.	431.
10 1/2	34.5	64.2	103.	145.	202.	272.	354.	449.
11	36.0	67.0	107.	151.	210.	284.	368.	466.
11 1/2	37.6	69.8	111.	158.	218.	295.	382.	484.
12	39.2	72.5	115.	164.	227.	306.	396.	501.
Heads.	1.8	5.8	11.1	13.6	22.6	39.0	58.0	83.5

For length of shaft required to form rivet-head, see p. 374.

NAILS.

Kinds.—The different kinds of nails may be classified as follows:

Wrought nails, which are forged either by hand labor or machine power· sometimes designated as *clinch nails,* on account of their property of bending without breaking. Seldom used in connection with wood-work, although they are the best *clinch* nail that can be had.

Cut nails, which are cut from a strip of rolled iron or steel of the thickness that the nail is to be and a little wider than the length of the nail. Cut nails are now commonly made of steel.

Wire nails, which are made from steel wire of the same size as the shank of the nail is to be.

Copper and brass nails—are manufactured, and are sometimes used in connection with marine and refrigerator work, and about physical laboratories, to avoid the magnetic effects of iron or steel.

Composition nails—are made of different alloys to avoid corrosion, or to prevent galvanic action set up by iron when in contact with zinc or other metals.

Varieties.—Nails are also made in a variety of shapes and sizes to adapt them to different classes of work; the principal varieties are indicated by the tables on following pages.

Galvanized-wire nails—may be obtained if desired. Wherever exposed to constant or frequent moisture they are more durable and satisfactory than uncoated nails, and are to be preferred for securing shingles, slates, and all kinds of roofing.

Cement-coated Nails.—J. C. Pearson Company, of Boston, Mass., have obtained a patent on coating wire nails with an asphaltum cement which greatly increases their holding power. Most varieties are carried in stock in the larger cities. With this coating slightly smaller nails may be used, with equal or greater holding power. It is claimed that it is cheaper for contractors to use cement-coated nails than ordinary wire nails. Most of the wooden-box factories use these nails, and they are especially desirable for nailing flooring, siding, etc.

Holding Power of Nails.—A committee appointed by the Wheeling nail manufacturers, a number of years ago, to test the comparative holding power of cut and wire nails, published the following data, although the kind of wood is not named.

POUNDS REQUIRED TO PULL NAILS OUT.

	Cut.	Wire.		Cut.	Wire.
Twenty-penny.......	1593	703	Sixpenny............	383	200
Tenpenny.........	908	315	Fourpenny..........	286	123
Eightpenny.........	597	227			

The following table shows the result of tests made at the U. S. Arsenal, Watertown, Mass., in 1902, the wood being pine:

COMPARATIVE ADHESIVE RESISTANCE OF COMMON SMOOTH WIRE NAILS AND CEMENT-COATED NAILS.

All nails driven into the same piece perpendicular to the grain.

Size and Name.	Diameter in Inches.	Length Driven,* Inches.	Adhesive Resistance,† Pounds.
Tenpenny, common, smooth	.145	2½	167
" coated	.117	2½	418
Ninepenny common, smooth	.132	2¼	182
" coated	.114	2¼	327
Eightpenny, common, smooth	.132	2	189
" coated	.112	2	316
Sixpenny, common, smooth	.097	1⅝	106
" coated	.092	1⅝	226

* All of the nails were left with their heads projecting from ¼ to ½ inch.
† Average of three trials.

The holding power of nails varies with the kind of wood into which they are driven. Austin T. Byrne gives the relative holding power of woods *about* as follows: White pine, 1; yellow pine, 1.5; white oak, 3; chestnut, 1.6; beech, 3.2; sycamore, 2; elm, 2; basswood, 1.2.

COMPARATIVE HOLDING POWER OF CUT AND WIRE NAILS.

Very thorough tests of the comparative holding power of wire nails and cut nails *of equal lengths and weights* were made at the U. S. Arsenal in 1892 and 1893. From forty series, comprising forty sizes of nails driven in spruce wood, it was found that the cut nails showed an average superiority of 60.50 per cent., the common nails showing an average superiority of 47.51 per cent. and the finishing nails an average of 72.22 per cent.

In eighteen series, comprising six sizes of *box* nails driven into pine wood, in three ways the cut nails showed an average superiority of 99.93 per cent. In no series of tests did the wire nails hold as much as the cut nails.

QUANTITY OF NAILS REQUIRED FOR DIFFERENT KINDS OF WORK.

For 1,000 shingles allow 5 lbs. fourpenny nails or 3½ lbs. threepenny.
1,000 laths, 7 lbs. threepenny fine, or for 100 square yards of lathing, 10 lbs. threepenny fine.
1,000 square feet of beveled siding, 18 lbs. sixpenny.
1,000 " " " sheathing, 20 lbs. eightpenny or 25 lbs. tenpenny.
1,000 " " " flooring, 30 lbs. eightpenny or 40 lbs. tenpenny.
1,000 " " " studding, 15 lbs. tenpenny and 5 lbs. twenty-penny.
1,000 " " " 1″ × 2½″ furring, 12″ centres, 9 lbs. eightpenny or 14 lbs. tenpenny.
1,000 " " " 1″ × 2½″ furring, 16″ centres, 7 lbs. eightpenny or 10 lbs. tenpenny.

CUT STEEL NAILS AND SPIKES.

SIZE, LENGTH, AND NUMBER TO THE POUND.

Ordinary.

Size.	Length, in Ins.	No. to Pound.
2*d*	$\frac{7}{8}$	716
3*d* fine	$1\frac{1}{16}$	588
3*d*	$1\frac{1}{4}$	448
4*d*	$1\frac{1}{2}$	336
5*d*	$1\frac{3}{4}$	216
6*d*	2	166
7*d*	$2\frac{1}{4}$	118
8*d*	$2\frac{1}{2}$	94
10*d*	3	72
12*d*	$3\frac{1}{8}$	50
20*d*	4	32
30*d*	$4\frac{1}{4}$	20
40*d*	$4\frac{3}{4}$	17
50*d*	5	14
60*d*	$5\frac{1}{2}$	10

Light.

Size.	Length, in Ins.	No. to Pound.
4*d*	$1\frac{3}{8}$	373
5*d*	$1\frac{3}{4}$	272
6*d*	2	196

Brads.

Size.	Length, in Ins.	No. to Pound.
6*d*	2	163
8*d*	$2\frac{1}{2}$	96
10*d*	$2\frac{3}{4}$	74
12*d*	$3\frac{1}{8}$	50

Clinch.

Length, in Ins.	No. to Pound.
2	152
$2\frac{1}{4}$	133
$2\frac{1}{2}$	92
$2\frac{3}{4}$	72
3	60
$3\frac{1}{4}$	43

Fence.

Length, in Ins.	No. to Pound.
2	96
$2\frac{1}{4}$	66
$2\frac{1}{2}$	56
$2\frac{3}{4}$	50
3	40

Spikes.

Length, in Ins.	No. to Pound.
$3\frac{1}{2}$	19
4	15
$4\frac{1}{2}$	13
5	10
$5\frac{1}{2}$	9
6	7

Boat.

Length, in Ins.	No. to Pound.
$1\frac{1}{2}$	206

Finishing.

Size.	Length, in Ins.	No. to Pound.
4*d*	$1\frac{3}{8}$	384
5*d*	$1\frac{3}{4}$	256
6*d*	2	204
8*d*	$2\frac{1}{2}$	102
10*d*	3	80
12*d*	$3\frac{5}{8}$	65
20*d*	$3\frac{7}{8}$	46

Core.

Size.	Length, in Ins.	No. to Pound.
6*d*	2	143
8*d*	$2\frac{1}{2}$	68
10*d*	$2\frac{1}{3}$	60
12*d*	$3\frac{1}{8}$	42
20*d*	$3\frac{3}{4}$	25
30*d*	$4\frac{1}{4}$	18
40*d*	$4\frac{3}{4}$	14
WH	$2\frac{1}{2}$	69
WHL	$2\frac{1}{4}$	72

Slate.

Size.	Length, in Ins.	No. to Pound.
3*d*	$1\frac{5}{16}$	288
4*d*	$1\frac{7}{16}$	244
5*d*	$1\frac{3}{4}$	187
6*d*	2	146

TACKS.

Size.	Length.	Number to Pound.	Size.	L'gth.	Number to Pound.	Size.	Length.	Number to Pound.
1 oz.	$\frac{1}{8}$	16,000	4 oz.	$\frac{7}{16}$	4,000	14 oz.	$\frac{13}{16}$	1,143
$1\frac{1}{2}$ "	$\frac{3}{16}$	10,066	6 "	$\frac{9}{16}$	2,666	16 "	$\frac{7}{8}$	1,000
2 "	$\frac{1}{4}$	8,000	8 "	$\frac{5}{8}$	2,000	18 "	$\frac{15}{16}$	888
$2\frac{1}{2}$ "	$\frac{5}{16}$	6,400	10 "	$\frac{11}{16}$	1,600	20 "	1	800
3 "	$\frac{3}{8}$	5,333	12 "	$\frac{3}{4}$	1,333	22 "	$1\frac{1}{16}$	727

STEEL-WIRE NAILS, SPIKES, AND TACKS.

SIZE, LENGTH, GAUGE, AND APPROXIMATE NUMBER TO THE POUND.

Compiled from Catalogue of American Steel and Wire Company, 1903.
Gauge is the A. S. and W. Co.'s Gauge, p. 1349.

Common Nails and Brads.*

Size.	Length, Inches.	Gauge.	No. to Pound.
2*d*	1	15	876
3*d*	$1\frac{1}{4}$	14	568
4*d*	$1\frac{1}{2}$	$12\frac{1}{2}$	316
5*d*	$1\frac{3}{4}$	$12\frac{1}{2}$	271
6*d*	2	$11\frac{1}{2}$	181
7*d*	$2\frac{1}{4}$	$11\frac{1}{2}$	161
8*d*	$2\frac{1}{2}$	$10\frac{1}{4}$	106
9*d*	$2\frac{3}{4}$	$10\frac{1}{4}$	96
10*d*	3	9	69
12*d*	$3\frac{1}{4}$	9	63
16*d*	$3\frac{1}{2}$	8	49
20*d*	4	6	31
30*d*	$4\frac{1}{2}$	5	24
40*d*	5	4	18
50*d*	$5\frac{1}{2}$	3	14
60*d*	6	2	11

Casing Nails.† / **Finishing Nails.†**

Casing Gauge.	Casing No. to Pound.	Finishing Gauge.	Finishing No. to Pound.
$15\frac{1}{2}$	1,010	$16\frac{1}{2}$	1,351
$14\frac{1}{2}$	635	$15\frac{1}{2}$	807
14	473	15	584
14	406	15	500
$12\frac{1}{2}$	236	$13\frac{1}{2}$	309
$12\frac{1}{2}$	210	13	238
$11\frac{1}{2}$	145	$12\frac{1}{2}$	189
$11\frac{1}{2}$	132	$12\frac{1}{2}$	172
$10\frac{1}{2}$	94	$11\frac{1}{2}$	121
$10\frac{1}{2}$	87	$11\frac{1}{2}$	113
10	71	11	90
9	52	10	62
9	46		
8	35		

Spikes.‡

Size.	Length, Inches.	Gauge.	No. to Pound.
10*d*	3	6	41
12*d*	$3\frac{1}{4}$	6	38
16*d*	$3\frac{1}{2}$	5	30
20*d*	4	4	23
30*d*	$4\frac{1}{2}$	3	17
40*d*	5	2	13
50*d*	$5\frac{1}{2}$	1	10
60*d*	6	1	8
7″	7	0	7
8″	8	00	6
9″	9	00	5
10″	10	$\frac{3}{8}$″	4
12″	12	$\frac{3}{8}$″	3

Shingle Nails.

Size.	Length, Inches.	Gauge.	No. to Pound.
3*d*	$1\frac{1}{4}$	13	429
4*d*	$1\frac{1}{2}$	12	274
5*d*	$1\frac{3}{4}$	12	235
6*d*	2	12	204
7*d*	$2\frac{1}{4}$	11	139
8*d*	$2\frac{1}{2}$	11	125
9*d*	$2\frac{3}{4}$	11	114
10*d*	3	10	83

Fine Nails.

Size.	Length, Inches.	Gauge.	No. to Pound.
2*d*	1	$16\frac{1}{2}$	1,351
3*d*	$1\frac{1}{8}$	15	778
4*d*	$1\frac{1}{2}$	14	473
3*d* extra fine	$1\frac{1}{8}$	16	1,015

* Common brads differ from common nails only in the head and point.
† Lengths are the same as common nails for corresponding size.
‡ Spikes are made with chisel points and diamond points; also with convex heads and flat heads.

STEEL-WIRE NAILS.—*Continued.*

Clinch Nails.				Fence Nails.*		Slating Nails.*	
Size.	Length, Inches.	Gauge.	No. to Pound.	Gauge.	No. to Pound.	Gauge.	No. to Pound.
2*d*	1	14	710			12	411
3*d*	$1\frac{1}{4}$	13	429	No. 5 s	mallest	$10\frac{1}{2}$	225
4*d*	$1\frac{1}{2}$	12	274	si	ze	$10\frac{1}{2}$	187
5*d*	$1\frac{3}{4}$	12	235	10	142	10	142
6*d*	2	11	157	10	124	9	103
7*d*	$2\frac{1}{4}$	11	139	9	92	Barbed Roofing Nails.†	
8*d*	$2\frac{1}{2}$	10	99	9	82		
9*d*	$2\frac{3}{4}$	10	90	8	62	$\frac{3}{4}'' \times$ No. 13	714
10*d*	3	9	69	7	50	$\frac{7}{8}'' \times$ No. 12	469
12*d*	$3\frac{1}{4}$	9	62	6	40	$1'' \times$ No. 12	411
16*d*	$3\frac{1}{2}$	8	49	5	30	$1\frac{1}{8}'' \times$ No. 12	365
20*d*	4	7	37	4	23	$1\frac{1}{4}'' \times$ No. 11	251

* Length same as clinch nails of corresponding size.

† Roofing nails are designated by the length, not by "penny." These nails are made up to 2 ins. long.

WIRE TACKS.

Title, Ounce.	Length, Inches.	Number per Pound.	Title, Ounce.	Length, Inches.	Number per Pound.	Title, Ounce.	Length, Inches.	Number per Pound.
1	$\frac{1}{8}$	16,000	4	$\frac{7}{16}$	4,000	14	$\frac{13}{16}$	1 143
$1\frac{1}{2}$	$\frac{3}{16}$	10,666	6	$\frac{9}{16}$	2,666	16	$\frac{7}{8}$	1,000
2	$\frac{1}{4}$	8,000	8	$\frac{5}{8}$	2,000	18	$\frac{15}{16}$	888
$2\frac{1}{2}$	$\frac{5}{16}$	6,400	10	$\frac{11}{16}$	1,600	20	1	800
3	$\frac{3}{8}$	5,333	12	$\frac{3}{4}$	1,333	22	$1\frac{1}{16}$	727
						24	$1\frac{1}{8}$	666

Wire carpet tacks are made polished, blued, tinned, or coppered; there are also upholsterers' and bill-posters' or railroad tacks

Expansion Bolts.—These are commonly used for bolting wood or iron to masonry that is already built. A hole is drilled in the masonry of such size that the expansion nut will fit closely, and when the bolt is screwed up the nut expands and binds firmly in the masonry. The illustration shows the Evans expansion bolt, which is also furnished with screw-head bolts. There are two other forms of expansion bolts on the market.

Expansion Bolt.

Screws.—Screws are made of iron, steel, brass, copper, bronze, and phosphor-bronze, the ordinary screw being of iron. Iron screws are finished with either a bright, blue, bronze, lacquered, tinned, or galvanized surface, and are also plated in nickel, brass, bronze, copper, and silver.

The size of screws is designated by the length in inches and the number of gauge—which denotes the diameter of the body of the screw. Thus a 1-in. No. 12 screw denotes a screw 1 in. long and .2158 of an inch in diameter.

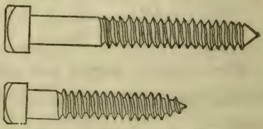
The gauge numbers range from No. 0 to No. 30, and the length from $\frac{1}{4}$ in. to 6 ins. Lengths vary by eighths of an inch up to 1 in., by quarters of an inch up to 3 ins., by halves up to 5 ins.

Screws from $\frac{5}{8}$ in. to $4\frac{1}{2}$ ins. long are made in about sixteen different gauge numbers.

The table on page 1346 shows the diameter to four places of decimals of the American Screw Gauge.

It should be noticed that unlike the ordinary wire gauges, the 0 of the screw gauge is the smallest, and the diameter increases with the number of the gauge.

Wood-screws are made with twenty-five different shapes of heads for different purposes. The most common shapes, however, are the ordinary flat head, round head, and oval head. The latter is tapered for counter-sinking, but is slightly rounded on top.



Lag- and Coach-screws.

Patent diamond-point steel screws are made especially for driving with a hammer.

Screws for metal have the same diameter throughout and the threads are V shaped.

Lag- or coach-screws are large heavy screws used where great

strength is required, as in heavy framing, and for fixing iron-work to timber. Lag-screws with conical point are made with diameters of 5/16, 3/8, 7/16, 1/2, 9/16, 5/8, 3/4, and 1 in., and in lengths from 1 1/2 to 12 ins.; coach-screws in diameters from 5/16 to 3/4 in. and in lengths from 1 1/2 to 12 ins.

HOLDING POWER OF LAG-SCREWS.

(Tests made by A. J. Cox, University of Iowa, 1891, quoted by Kent, page 290.)

Kind of Wood.	Size Screw.	Size Hole Bored.	Length in Wood.	Maximum Resistance, Lbs.	No. Tests.
	Inch.	Inch.	Inches.		
Seasoned white oak.......	5/8	1/2	4 1/2	8,037	3
" " "	9/16	7/16	3	6,480	1
" " "	1/2	3/8	4 1/2	8,780	2
Yellow-pine stick.........	5/8	1/2	4	3,800	2
White cedar, unseasoned. .	5/8	1/2	4	3,405	2

(Hoopes & Townsend give the force at which screws were drawn out of yellow pine as follows:)

Screw....	1/2 in.	5/8 in.	3/4 in.	7/8 in.	1 in.
Wood, depth..............	3 1/2 ins.	4 ins.	4 ins.	5 ins.	6 ins.
Force, pounds............	4,960	6,000	7,685	11,500	12,620

Wood-screws are sold by the gross, lag- and coach-screws by the pound.

DATA ON EXCAVATING.

Excavating is almost invariably measured by the cubic yard of 27 cu. ft.

For measuring excavations of irregular depth see p. 69.

For computing the contents of wells and cesspools, the circular area in square feet may be obtained from the table on p. 53, and this multiplied by the depth in feet will give contents in cubic feet.

The cost of excavating and removing earth is ordinarily made up of the following items:

a. Loosening the earth for the shovellers.

b. Loading by shovels into carts or barrows.

c. Hauling or wheeling it away, including emptying and returning.

d. Spreading it out on the dump.

For every large job, such as railroad work, it is also necessary to make an allowance for keeping the hauling road in repair, sharpening and repair of tools, carts, harness, superintendence, and water-carriers.

Where the dirt excavated can be spread over the ground immediately surrounding the excavation the loosened dirt may be removed by scrapers without shovelling.

Data for Estimating Cost.—For loosening: Two men with a plough and team of horses will loosen from 20 to 30 cu. yds. of strong, heavy soils per hour or from 40 to 60 cu. yds of ordinary loam. One man with a pick will loosen $1\frac{1}{3}$ yds. per hour of stiff clay or cemented gravel, 4 yds. of common loam, or 6 yds. of light sand.

The average quantity of *loosened earth* which a man can shovel into a cart per hour is:

Loam or sand	2.0 cu. yds.
Clay and heavy soils	1.7 "
Rock	1.0 cu yd.

Average earth loosened swells to from $1\frac{1}{5}$ to $1\frac{1}{3}$ times its original bulk in place.

The capacity of vehicles used for moving excavated materials is about as follows:

Wheelbarrows	3 to 4 cu. ft.
1-horse dump-carts	18 " 22 "
2-horse dump-wagons	27 " 45 " *
Drag scrapers	3 " 7 "
Wheel scrapers	10 " 17 "
Dump-cars on rails	27 " 80 "

The economical length of haul with drag scrapers is about 150 ft.; with wheeled scrapers, 500 ft.; with wheelbarrows, 250 ft.; with 1-horse dump-carts, 600 ft.†

The average speed of horses is given as about 200 ft. per minute.

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* The ordinary load for 2-horse wagons such as are commonly used for hauling dirt, sand, and gravel is from $1\frac{1}{4}$ to $1\frac{1}{2}$ cu. yds.

† Inspectors' Pocket-book, A. T. Byrne, C E.

Much valuable data for estimating the cost of excavating may be found in Trautwine's Engineer's Pocket-book, p. 800–810.

Weight of Earth, Sand, and Gravel.—For general calculations the following average values may be taken:

14 cu. ft. of	chalk	weigh	1	ton	19 cu. ft. of	gravel	weigh	1	ton
18 " "	clay	"	1	"	22 " "	sand	"	1	"
21 " "	earth	"	1	"					

Rock Excavation.—A cubic yard of rock, in place, when broken up by blasting for removal by wheelbarrows or carts, will occupy a space of about $1\frac{4}{5}$ cu. yds.; consequently the cost of hauling or removal is about 50 per cent. more than for dirt.

With labor at $1 per day, the actual cost for loosening hard rock, including tools, drilling, powder, etc., will average about 45 cts. per cubic yard, in place, under all ordinary circumstances. In practice it will generally range between 30 and 60 cts., depending on the position of the strata, hardness, toughness, water, and other considerations. Soft shales and other allied rocks may frequently be loosened by pick and plough as low as 15 to 20 cts., while on the other hand shallow cuttings of very tough rock with an unfavorable position of strata, especially in the bottoms of excavations, may cost $1 per cubic yard, or even considerably more. The quarrying of average hard rock requires about $\frac{1}{4}$ to $\frac{1}{3}$ lb. of powder per cubic yard, in place, but the nature of the rock, the position of the strata, etc., may increase it to $\frac{1}{2}$ lb. or more. Soft rock frequently requires more powder than hard. A good churn-driller will drill 8 to 10 ft. in depth of holes about $2\frac{1}{2}$ ft. deep and 2 ins. diameter per day in average hard rock, at from 12 to 18 cts per foot.*

DATA ON STONEWORK.

(For description of various kinds of stonework, see Building Construction and Superintendence, Part I Chapter VI.)

The commonest kind of stonework, *i.e.* for walls, is called *rubblework.* No work whatever is done on the stones except to break them up with a hammer.

If the wall is built in courses it is designated as coursed rubble.

* Trautwine, p. 810.

When the stones showing on the outside face of the wall are squared, the work is designated as *ashlar.* Ashlar is of two kinds: *coursed ashlar*, in which the stones are laid to form courses around the building, all of the stones in any course being of the same height, and *broken ashlar*, in which stones of different heights are used. *Hammer-dressed* ashlar designates work where the stones are roughly squared with a hammer. This is a very cheap class of work. Good ashlar work should be squared on the bench with chisels, and with beds and end joints cut square to the face.

Stonework which requires a chisel or any other tool except a hammer for dressing is called "cut-work." Cut-work costs considerably more than hammer-dressed work.

Measurement of Stonework.—Rough stone from the quarry is usually sold under two classifications: rubble- and dimension-stone. Rubble includes the pieces of irregular size most easily obtained from the quarry, and suitable for cutting into ashlar 12 ins. or less in height and about 2 ft. long. Stone ordered of a certain size, or to square over 24 ins. each way, and of a particular thickness, is called dimension-stone. The price of the latter varies from two to four times the price of rubble.

Rubble is generally sold by the ton or car-load. Footings and flagging are usually sold by the square foot; dimension-stone by the cubic foot. In Boston granite blocks for foundations are usually sold by the ton.

In estimating on the cost of stonework put into a building, the custom varies with different localities, and even among contractors in the same city.

Dimension-stone footings (that is, squared stone 2 ft. or more in width) are usually measured by the square foot. If built of large rubble or irregular stones the footings are measured in with the wall, allowance being made for the projections of the footings.

Rubblework is almost universally measured by the perch of $16\frac{1}{2}$ cu. ft. The author has been unable to find any locality where the legal perch of $24\frac{3}{4}$ cu. ft. is used by stone-masons. In Philadelphia, St. Louis, and some portions of Illinois, 22 cu. ft. are called a perch.

Railroad work is usually measured by the cubic yard.

When stonework is let by the perch, the number of cubic feet to the perch should be stated in the contract, and also whether or not openings are to be deducted. As a rule no

deductions are made for openings of less than 70 superficial feet.

Data for Estimating Cost.—The price of common rubble as it comes from the quarry will vary from 50 cts. to \$1.50 per ton, f.o.b. at point of delivery, according to the cost of quarrying, transportation, etc. \$1.25 a perch is probably a fair average.

A ton of most stones will make from 1 to $1\frac{1}{4}$ perch.

The cost of laying one perch of stone may be estimated by the following items:

Labor: mason $2\frac{3}{8}$ hrs., helper $1\frac{2}{3}$ hrs. (based on 2 helpers to 3 masons); sand $\frac{1}{5}$ load; lime $\frac{3}{4}$ bu., or if laid in all cement mortar, one perch will require from $\frac{1}{3}$ to $\frac{1}{2}$ bbl. cement.

At average wages, rubble cellar walls, 18 ins. to 2 ft. thick, laid in lime mortar, vary in cost from \$2.75 to \$4 per perch, \$3.25 a perch being a fair average; in all cement mortar from \$3.25 to \$4.25 per perch.

The cost of ashlar depends very largely upon the kind of stone used and the distance it has to be brought. The price of the rough stock on the cars at point of delivery may vary from 70 cts. to \$1.25 per cubic foot for granite and 55 cts. to \$1 for sandstones and limestones, depending largely upon cost of transportation. 1 cu. ft. of stone should make 2 sq. ft. of ashlar, at least. Some quarries get out stone especially suitable for ashlar and sell it at about 25 cts. per lineal foot for courses 12 ins. high.

The cost of cutting ashlar, with stone-cutters' wages at \$4 per day, will average about 15 cts. per square foot for soft stones, 15 to 20 cts. per square foot for hard sandstones and limestones, and 25 to 30 cts. for granite. The cost of setting ashlar will vary from 10 cts. per square foot to 25 cts. for soft stones or 30 cts. for granite, 15 cts. being an average price for sandstones and limestones.

The cost of cut-stone trimmings depends so largely upon the kind of stone, that it is quite impossible to give prices that would be of any service.

The following figures, however, quoted from The Building Trades Pocket-book, may be of some guide in forming a rough estimate, the prices if anything being probably a little above the cost of the local stone in most localities.

Flagstones for sidewalks, ordinary stock, natural surface, 3 ins. thick, with joints pitched to line, in lengths (along walk) from 3 to 5 ft., will cost, for 3-ft. walk, about 8 cts. per square

foot (if 2 ins. thick, 6 cts.); for 4-ft. walk, 9 cts.; and for 5-ft. walk, 10 cts. per square foot. The cost of laying all sizes will average about 3 cts. per square foot. The above figures do not include cost of hauling.

Curbing (4-in.×24-in. granite) will cost at quarry from 25 to 30 cts. per lineal foot; digging and setting will cost from 10 to 12 cts. additional; and the cost of freight and hauling must also be added.

The following figures show the approximate cost of cut bluestone for various uses:

Flagstone, 5 ins., size 8 ft.×10 ft., edges and top bush-hammered, per square foot face measure	$0.65
Flagstone, 4 ins., size 5 ft.×5 ft., select stock, edges clean-cut, natural top, per square foot	.30
Door-sills, 8 in.×12 in.. clean cut, per lineal foot	1.25
Window-sills, 5 in.×12 in., clean-cut, per lineal foot	.80
Window-sills, 4 in.×8 in., clean-cut, per lineal foot	.45
Window-sills, 5 in.×8 in., clean-cut, per lineal foot	.60
Lintels, 4 in.×10 in., clean-cut, per lineal foot	.60
Lintels, 8 in.×12 in., clean-cut, per lineal foot	1.10
Water-table, 8 in.×12 in., clean-cut, per lineal foot	1.25
Coping, 4 in.×21 in., clean cut, per lineal foot	1.10
Coping, 4 in.×21 in., rock-face edges and top, per lineal foot	.45
Coping, 3 in.×15 in., rock-face edges and top, per lineal foot	.25
Coping, 3 in.×18 in., rock-face edges and top, per lineal foot	.30
Steps, sawed stock, 7 in.×14 in., per lineal foot	.90
Platform, 6 in. thick, per square foot	.45

To the prices of cut stone above given must be added the cost of setting, which, for water-tables, steps, etc., will be about 10 cts. per lineal foot, and for window-sills, etc., about 5 cts. per lineal foot.

DATA ON BRICKS AND BRICKWORK.

[For a complete description of clay bricks, their process of manufacture, etc., also of all kinds of brickwork, see Chapter VII, Part I, of Building Construction and Superintendence.]

The word brick as commonly used refers to blocks made from clay that have been moulded into the required shape and burned in a kiln, and until quite recently practically all bricks were made from clay; at the present time, however, bricks are also made from sand and lime.

Clay Bricks.—These may be broadly classified as common brick, face-brick, fire-brick, and paving-brick.

As to the process of manufacture, bricks are classified as

soft-mud bricks, stiff-mud bricks, dry-pressed bricks, and re-pressed bricks.

Soft-mud bricks are made by tempering the clay with water until it becomes soft and plastic, when it is pressed into the moulds either by hand or by a machine. Practically all hand-made bricks are soft-mud bricks.

Stiff-mud bricks are machine-made. The clay is first ground and only enough water is added to make a stiff mud. The stiff clay is forced through a die or dies in the machine in a continuous stream, which is cut up automatically into pieces the size either of the end or side of the brick. If the opening is the size of the end of the brick, the bricks are end-cut; if of the size of the side of the brick, they are side-cut. Stiff-mud bricks can readily be distinguished from soft-mud bricks by their appearance. As good if not better bricks can be made by the soft-mud process as by the stiff-mud process, and in the Eastern States the soft-mud bricks are probably the strongest. As far as the author's observation has extended in the Western States, the stiff-mud bricks are as a rule preferable to those made by the soft-mud process.

Stiff-mud bricks are usually heavier than soft-mud or hand-made bricks.

Soft-mud bricks are often *re-pressed* to make face-bricks.

Dry-pressed bricks are made almost entirely for face-work, although in some localities dry-pressed bricks are also used for common bricks. Hydraulic-pressed bricks are dry-pressed. *Moulded bricks* are always dry-pressed. Very fine bricks are made by this process.

Bricks made by any of the above processes require to be burned in a kiln. According to their position in the kiln, common brick are designated as *arch* or hard-burned brick, *red* or well-burned brick, and *salmon* or soft brick. As a rule, salmon brick are not fit to use in an exterior or bearing wall.

Color.—The color of brick depends principally upon the presence of iron, lime, and magnesia in the clay. A large proportion of oxide of iron gives a clear bright red. Magnesia produces a brown color, and when in the presence of iron gives a light drab color.

Dry-pressed bricks are often colored artificially either by mixing clays of different composition, or by mixing mineral colors with the finely ground clay.

Fire-bricks are ordinarily made from a mixture of flint

clay and plastic clay. They are usually white or white mixed with brown in color and are used for the lining of furnaces, fireplaces, and tall chimneys.

Paving-bricks are a very hard brick, usually vitrified or annealed. They are much more expensive than common brick and are seldom used in buildings.

Size and Weight of Clay Bricks.—In this country there is no legal standard for the size of bricks, and the dimensions vary with the maker and also with the locality. In the New England States the common brick averages about $7\frac{3}{4}\times3\frac{3}{4}\times2\frac{1}{4}$ ins. In most of the Western States common bricks measure about $8\frac{1}{2}\times4\frac{1}{8}\times2\frac{1}{2}$ ins., and the thickness of the walls measures about 9, 13, 18, and 22 inches for thickness of 1, $1\frac{1}{2}$, 2, and $2\frac{1}{2}$ bricks. The size of all common bricks varies considerably in each lot, according to the degree to which they are burnt; the hard bricks being from $\frac{1}{8}$ to $\frac{3}{16}$ of an inch smaller than the salmon bricks.

Pressed bricks or face-bricks are more uniform in size, as most of the manufacturers use the same size of mould. The prevailing size for pressed bricks is $8\frac{3}{8}\times4\frac{1}{8}\times2\frac{3}{8}$ ins. Pressed bricks are also made $1\frac{1}{2}$ ins. thick and $12\times4\times1\frac{1}{2}$ ins., the latter size being generally termed Roman brick, or tile.

The weight of bricks varies considerably with the quality of the clay from which they are made, and also, of course, with their size. Common bricks average about $4\frac{1}{2}$ lbs. each, and pressed bricks vary from 5 to $5\frac{1}{2}$ lbs. each.

For strength of bricks and brickwork, see pp. 213, 218, 229.

Fire-bricks are made in various forms to suit the required work. A straight brick measures $9\times4\frac{1}{2}\times2\frac{1}{2}$ ins. and weighs about 7 lbs.

To secure the best results fire-bricks should be laid in the same clay from which they are manufactured, this being mixed with water into a thin paste. The thinner the joint, the better the wall will stand heat.

Paving-bricks vary in size and weight according to the locality and the requirements of the specifications.

The "standard" bricks are $2\frac{1}{2}\times4\times8$ ins., requiring 61 bricks to the square yard (on edge), and weigh 7 lbs. each. "Repressed" bricks are $2\frac{1}{2}\times4\times8\frac{1}{2}$ ins., requiring 58 to the square yard and weigh $6\frac{1}{2}$ lbs. each. "Metropolitan" are $3\times4\times9$ ins., requiring 45 to the square yard, and weigh $9\frac{1}{2}$ lbs. each.*

* Building Inspectors' Pocket-book.

Sand-lime Brick.—Bricks made of sand and lime have been made in Germany for fifty years or more and the industry appears to be established on a successful basis. During the past three or four years a number of plants have been equipped in this country for the manufacture of these bricks. There are three or four different processes of manufacture, the principal ones being the "Huennekes" and "Schwarz" systems. Parties who have personally inspected plants operated by each system appear to be divided in their opinion as to which process produces the best brick. The hardening process, by steam, is common to all systems, as the practicability of sand-lime bricks is due to the formation of silicate of lime, brought about by the heat and moisture.

Under the "Huennekes System" the process is briefly as follows:* The sand is put through a dryer, and then passes to a measuring-machine. The lime, previously burnt in a kiln, is crushed very fine, and then passes to the measuring-machine, and is mixed with the sand just after leaving it. The mixture is measured to contain 94 per cent. of sand and 6 per cent. of lime. This mixture is then conveyed to a tube-mill, where it is ground very fine, and then drops into the wet mixer, where enough water is added so that the mixture will ball easily in the hand. It is then conveyed to a large bin, where it is kept about twelve hours to permit the lime to slake. From this bin it goes to the press in which the bricks are formed under a pressure of about 175 tons to the brick. They are then piled on cars holding from 850 to 1000 bricks, and the cars are run into a steel cylinder 62 ft. long and 6 ft. in diameter, fitted with a track and a tank for holding chemicals. As soon as the cylinder is filled, the head is bolted on and steam is introduced. The steam on entering goes through the chemical tank and becomes supercharged with the chemical. Steam is kept at 120 lbs. pressure for eleven hours, when it is blown off, the head taken off, and the bricks are taken out ready for the market.

The chemical combination is controlled exclusively by H. Huennekes Company of New York City, who license and equip factories for the manufacture of bricks under their process.

The Schwarz system † differs from the above in the preparation of the sand and lime for the press, which is done in a prepar-

* This information was furnished the author by Mr. D. P. De Long, Prest. Granite Brick Company of Glens Falls, N. Y.

† Schwarz System Brick Co., 8 Bridge Street, New York.

ing-machine invented by Dr. Schwarz of Zurich, Switzerland; also no chemicals are used with this system—nothing but lime and sand. [It is claimed that not a single sand-lime brick factory in Europe applies any kind of chemicals whatsoever.] In the Schwarz preparing-machine all the moisture of the sand is first removed by drying the same under vacuum; the lime is then added and the materials thoroughly mixed by two wing-shaped agitators revolving in the cylinder in opposite directions. Then follows a carefully measured and always constant amount of water to slaken the lime, and the heat evolved upon this reaction is immediately utilized for a second reaction, i.e., the opening up of the silica of the sand and the formation of silicate of lime, by which process the mass is rendered soft and plastic, easy to mold, and causing little wear and tear to the press. Finally, the mixing going on continually, again a definite amount of water is added to moisten the mass for the press, whereupon the apparatus is emptied and recharged.

The advocates of the Schwarz system claim that their process produces a more uniform mixture, and consequently a brick of more uniform quality.

The sand used should be a sharp silica sand free from clay * and nearly free from loam. The lime should be a fat, quick-slaking lime free from magnesia.

Qualities.—The natural color of sand-lime brick is white or a light gray; the bricks are said to present a fine appearance. They are very dense, and show a very small absorption of moisture, usually under 10 per cent.

The average crushing strength seems to be about 3,000 lbs. per square inch, although tests have shown an ultimate strength of 6,700 lbs. per square inch.

The bricks made by the Granite Brick Company of Glens Falls, N. Y., measure $8\frac{1}{4} \times 4 \times 2\frac{1}{4}$ ins., and weigh 5 lbs. each. This company is selling its face brick at $10 per M., f.o.b. at factory.

The author understands that sand-lime bricks are being used to a considerable extent in different portions of the country.

Glazed and Enamelled Brick.†—The terms "enamelled brick" and "glazed brick," as commonly used, refer practi-

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* The presence of clay in the sand tends to prevent its standing freezing weather.

† For description of process of manufacture, see p. 197 Building Construction and Superintendence, Part I.

cally to the same article, and neither include what is known as a "salt-glazed" brick. The enamelled or glazed brick are generally dipped or sprayed and then burned, whereas the "salt-glaze" is obtained by the introduction of salt into the fire-boxes of kilns while the bricks are being burned. Glazed or enamelled brick are generally divided into two classes: true enamelled brick, which has a glaze containing the coloring matter applied to it without any intermediate slip; the other has a transparent glaze placed over a white or colored slip, the slip coming between the glaze and the material to be glazed. The latter is the process most used in this country. Manufacturers differ as to which process produces the best brick, although it would seem as though the true enamel would not chip or "peel" as readily. These bricks can be made in a variety of colors, from white to dark green or chocolate, and either in a *highlg glazed or satin* (*dull finish*), the latter finish being quite desirable in many instances on account of its doing away with the glare of the more highly glazed bricks or tile.

An enamelled surface may be distinguished from a glazed surface by chipping off a piece of the brick. The glazed brick will show the layer of slip between the glaze and the brick; the enamelled brick will show no line of demarcation between the body of the brick and the enamel.

Enamelled bricks are made in two regular sizes, English size ($9'' \times 3''$ enamelled surface, $4\frac{1}{2}'$ bed) and American size ($8\frac{3}{8}'' \times 2\frac{1}{4}''$ enamelled surface, $4\frac{1}{8}''$ bed).

The English size costs about $10 per M. more than the American, but on account of the saving in the number of bricks, labor of laying, and mortar in joints, they really effect a saving of about 7 cts. per square foot.

The Tiffany Enamelled Brick Company also make a "Norman flat" ($12'' \times 4\frac{1}{8}''$ enamelled surface, $2\frac{1}{4}''$ bed).

The selling price of enamelled brick in Chicago at the present time (June, 1904), is as follows:

American size ..	$75 per M.	English size.....	$85 per M.
Norman flat	$100 "		

At these prices the cost of the bricks per square foot will be:

American size, 7 bricks to the foot	$52\frac{1}{2}$ cts.
English size, $5\frac{1}{3}$ bricks to the foot	$45\frac{1}{3}$ "
English flat, $3\frac{3}{5}$ bricks to the foot.........	36 "
Norman flat, 3 bricks to the foot..........	30 "

The standard colors carried in stock are white, cream, and buff; other colors are made to order.

American enamelled and glazed bricks are now extensively used for the exterior surfaces of buildings, particularly for street fronts, light courts, and for interior side walls and partitions of rooms or buildings used for a great variety of purposes.

The principal manufacturers are the Tiffany Enamelled Brick Company, Chicago; Blue Ridge Enamelled Brick Company, Newark, N. J.; Pennsylvania Enamelled Brick Company, New York City.

Estimating Quantities and Cost of Brickwork.

The almost universal method of figuring the cost of brickwork is by estimating the number of thousands of bricks, wall measure, and then multiplying by a certain price per thousand, which is usually determined by experience and which is intended to include every item affecting the cost, and very often the profit. All of the common brickwork in any given building is usually figured at the same price per thousand, the adjustment for the more expensive portions of the work being made in the manner of measuring.

The principle underlying this system is explained as follows:

"The plain dead wall of brickwork is taken as the standard, and the more difficult, complicated, ornamental, or hazardous kinds of work are measured up to it so as to make the compensation equal.

"To illustrate: If, in one day, a man can lay two thousand bricks in a plain dead wall, and can lay only five hundred in a pier, arch, or chimney-top in the same time, the cost of labor per thousand in such work is four times as much as in the dead wall, and he is entitled to extra compensation; but instead of varying the price, the custom is to vary the measurement to compensate for the difference in the time, and thus endeavor to secure a uniform price per thousand for all descriptions of ordinary brickwork, instead of a different price for the execution of the various kinds of work."*

Wall Measure, How Figured.—Plain walls are quite universally figured at 15 bricks to the square foot of 8- or 9-in. wall, 22½ bricks per square foot of 12- or 13-in. wall, 30 bricks per square foot of 16- or 17-in. wall, and 7½ bricks for each additional 4 or 4½ ins. in thickness of the wall. These figures

* From Rules of Measurement adopted by the Brick Contractors' Exchange of Denver, Col.

are used without regard to the size of the bricks, the effect of the latter being taken into account in fixing the price per thousand. No deduction is made for openings of less than 80 superficial feet, and when deductions are made for larger openings the width is measured 2 ft. less than the actual width. Hollow walls are also measured as if solid. To the number of bricks thus obtained is added the measurement for piers, chimneys, arches, etc.

Footings are generally measured in with the wall by adding the width of the projection to the height of the wall. Thus if the footings project 6 ins. on each side of the wall, 1 ft. is added to the actual height of the wall.

Chimney-breasts and pilasters are measured by multiplying the girth of the breast or pilaster from the intersections with the wall by the height, and then by the number of bricks corresponding with the thickness of the projection. Flues in chimneys are always measured solid.

Detached chimneys and chimney-tops are measured as a wall having a length equal to the sum of the side and two ends of the chimney, and a thickness equal to the width of the chimney. Thus a chimney measuring 3 ft. by 1 ft 4 ins. would be measured as a 16- or 17-in. wall, 5 ft. 8 ins. long.

The rule for independent piers is to multiply the height of the pier by the distance around it in feet, and consider the product as the superficial area of a wall whose thickness is equal to the width of the pier. In practice, many masons measure only one side and one end of a pier or chimney.

Arches of common bricks over openings of less than 80 superficial feet are usually disregarded in estimating. If the arch is over an opening larger than 80 sq. ft., the height of the wall is measured from the springing line of the arch. No deduction is made in the wall measurement for stone sills, caps, or belt courses, nor for stone ashlar, if the same is set by the brick-mason. If the ashlar is set by the stone-mason, the thickness of the ashlar is deducted from the thickness of the wall.

The sum of all of these measurements represents a certain number of thousands of bricks, and the whole is then multiplied by a common price per thousand, as $6, $8, $12, or $16, according to whatever the cost of plain brickwork may be. If the building is to be faced with **pressed brick,** *the actual cost* of the pressed brick, as nearly as it can be computed, is added to the estimated price of the common brick work, nothing

being added for laying the pressed brick, nor anything deducted from the common-brick measurement, the measurement of the common work displaced by the pressed brick being assumed to offset the difference in the cost of laying the pressed and common brickwork.

In arriving at the cost of the pressed brick, the external superficial area of the walls faced with such brick is computed, and all openings, belt courses, stone caps, etc., deducted. 5-in stone sills are not usually deducted. If a portion of the wall is covered by a porch, so that common brick may be used back of it, this space is also deducted. The net pressed-brick surface is then multiplied by 6, 6½, or 7 to obtain the number of bricks required, 6½ giving about the number of pressed bricks required to the square foot of the standard size.

The topping out of the chimneys, if of face-brick, is measured by girting the chimney and multiplying by the height, and adding the sum to the wall area.

Example.—As a simple example of this system of estimating we will take a small brick house 28 by 32 ft. without cross-walls, the basement walls to be 13 ins. thick, with footings 2 ft. 6 ins. wide; first-story walls, 13 ins. thick; second-story walls, 9 ins. thick; height of basement walls from trench to top of first-floor joists, 8 ft. 6 ins.; from first-floor joists to top of second-floor joists, 10 ft. 6 ins.; from second-floor joists to plate, 9 ft.

Wall Measurement.—Basement walls, equal 120 ft. (girth of building) ×9 ft. 10 ins. (height and projection of footing) ×22½; equals 26,550 bricks.

First-story walls, 120 ft.×10 ft. 6 ins.×22½; equals 28,360 bricks.

Second-story walls, 120 ft.×9 ft.×15; equals 16,200 bricks.

Topping out two chimneys, each 1 ft. 9 ins.×1 ft. 5 ins., 14 ft. high above roof, equals 2×14 ft.×(1 ft. 5 ins.+1 ft. 9 ins. +1 ft. 5 ins.)×30; equals 3,600 bricks.

Total brickwork equals 74,710 bricks; at $9 per M. (present price in Denver), equals $672.39.

Pressed Brick.—From grade to the under side of plate the wall measures 22 ft. 6 ins.; to be faced with $15 pressed brick of the standard size.

The door and window openings measure 384 superficial feet.

Surface of pressed brick equals $120 \times 22\frac{1}{2}$, equals . .	2,700	sq. ft.
Deduct for openings.	384	"
	2,316	"
Add for two chimneys, $2 \times 14 \times 6$ ft. 4 ins., equals. . .	177	"
	2,493	"

$2{,}493 \times 6\frac{1}{2}$ equals 16,204 pressed bricks, at $15 per M., equals $243.

Total amount of bid, $672.39 + $243, equals $915.39.

The above figures are supposed to include the necessary lime, sand, water, scaffolding, etc., required to make the mortar and put up the walls, and also a profit for the contractor, but anything in the way of ironwork, as ties, thimbles, ash doors, etc., are figured additional to the above.

Detailed Estimates. — In estimating by the above method, the price per thousand is to some extent a matter of guesswork, and while an experienced contractor may perhaps make as accurate an estimate by this method as is possible by any, yet it is often necessary to estimate the work in detail, and even when the work has been estimated as above, it is necessary for the contractor to know how many bricks and how much sand and lime will be required to do the work. The following data will assist in making such detailed estimates:

With the size of bricks used in the Western States, from $16\frac{1}{2}$ to $17\frac{2}{3}$ common bricks are required to the cubic foot after deducting openings, and figuring the thickness of walls at 8, 12, 16, 20 ins., etc., or the actual number of bricks required will run about two-thirds of the " wall measure " when the openings are of about the average number and size.

The number of pressed bricks will be about 6 or $6\frac{1}{2}$ bricks to the foot after deducting openings.

To lay 1,000 common bricks, kiln count, requires $2\frac{1}{2}$ bushels or 200 lbs. of white lime and $\frac{5}{8}$ yd. of sand. For a good lime and cement mortar allow 2 bushels lime, 1 bbl. cement, and $\frac{5}{8}$ yd. sand. For 1 to 3 cement and sand mortar allow $1\frac{1}{2}$ bbls. cement and $\frac{5}{8}$ yd. sand, or one half load.

To lay 1,000 pressed bricks with buttered joints will require 2 bushels of lime (160 lbs.) and $\frac{1}{4}$ yd. of sand; with spread joints 2 to $2\frac{1}{2}$ bushels of lime and $\frac{3}{8}$ to $\frac{1}{2}$ yd. of sand.

If colored mortar is used, about $1 per 1,000 bricks should be added for the mortar color.

A brick-mason, working on a city job under a good foreman, will lay 60 pressed (face) bricks per hour, on an average, and from 150 to 175 common bricks per hour, 160 being a fair average. In country towns the average is nearer 120 an hour.

With wages at 62½ cts. per hour for masons, 31¼ cts. for hod-carriers, and 34⅜ cts. for mortar-mixers and carriers, sand at 60 cts. a yard, and lime at 40 cts. per bushel of 80 lbs., brick-masons in Denver find that the average cost for laying common brick in 12-in. walls is about \$6 per M., kiln count, and for laying pressed brick, about \$10 per M.

For common brickwork, one helper will be required for every mason, and on 9-in. walls faced with pressed brick, one helper to every two masons.

In building common-brick fireplaces and chimneys one mason and helper will lay about 600 bricks in a day of nine hours.

As a rule, chimneys of common brick with 4-in. walls cost about 50 cts. per running foot, in height, for single flues, and 90 cts. for double flues.

Space Required for Piling Bricks.—One thousand bricks closely stacked occupy about 56 cu. ft.

One thousand old bricks, cleaned and loosely stacked, occupy about 72 cu. ft.

A brick-layer's hod measures 21 ins. × 7 ins. × 7 ins., and will hold 18 bricks.

A mortar hod measures 24 ins. × 12 ins. × 12 ins. × 12 ins. across the top.

Mortar colors are usually in the form of a dry powder, put up in barrels, the number of pounds to the barrel and price per pound averaging about as follows:

Red, in 500-lb. barrels, dry.	2 cts. per pound
Brown, in 450-lb. barrels, dry.	2½ " " "
Buff, in 400-lb. barrels, dry.	2½ " " "
Black, in 1,000-lb. barrels, dry.	3½ " " "

Red, brown, buff, or black, in pulp, ¼ ct. per pound extra. For lots of less than full barrels an extra charge is made for packing and drayage.*

To color the mortar for laying 1,000 bricks with spread joints will require about 50 lbs. of red, brown, or buff, and from 40 to 45 lbs. of black; with buttered joints, 40 lbs. of red, brown, or buff, or from 25 to 35 lbs. of black.

The colors should first be mixed with dry sand, then the

* These figures are for Ricketson's mortar colors.

cold slaked lime added and again mixed thoroughly. It is very important that the color be uniformly mixed. If it is not added *at first*, but is left until the mortar is made, the labor of mixing is doubled. The more thorough the mixing the less color is required. Mortar colors should *never be mixed with hot lime.*

LIME.

Definitions and Useful Data.—Pure lime is a protoxide of calcium, or, in other words, a metallic oxide. It has a specific gravity of 2.3, is amorphous, somewhat spongy, highly caustic, quite infusible, possesses great affinity for water, and if brought in contact with it will rapidly absorb 22 to 23 per cent. of its weight, passing into the condition of hydrate oı lime.

Slaked lime is hydrate of lime.

Quicklime, or caustic lime, is the resulting lime left from the calcination of limestone. It is chemically known as calcium oxide.

Limestone, carbonate of lime. *Crystallized lime*, marble. *Fossil lime*, chalk. *Sulphate of lime*, gypsum. *Calcination* is heating to redness in air.

Slaking is the process of chemical combination of quicklime with water.

Air-slaking.—Hydration by the absorption of moisture from the atmosphere.

Lime is shipped either in barrels or bulk. In dry climates it will keep for a long time in bulk, but in damp climates and along the coast it soon slakes unless enclosed in barrels.

In most of the Eastern cities it is sold by the barrel, weighing for Rockland (Me.) lime 220 lbs. net. When shipped in bulk it is generally sold by the bushel of 80 lbs., 2½ bushels, or 200 lbs., of lime being considered as equivalent to a barrel.

The average yield of *lime paste* from the best Eastern limes has been found to be 2.62 times the bulk of unslaked lime. A barrel of good quality well-burnt lime should make 8 cu. ft., or 20 pails, of lime paste or putty.

Careful experiments conducted by U. S. engineers have demonstrated that the best mortar is obtained by mixing one part lime paste to two of sand.

"Popping" of Lime.—The best qualities of lime should completely slake in forty-eight hours, but there are some limes in which some of the particles will not slake with the bulk of the lime, but continue to absorb moisture, and finally, after a long

period extending sometimes over two years, they will slake or "pop," and if on a wall or ceiling will cause a speck of plaster to fall off. Such limes should not be used for plastering.

Cements.—For data on cements, see pp. 192–204.

SAND AND GRAVEL.

Sand is obtained from banks or pits, from river-beds, and from the seashore. Pit- or bank-sand, free from clay or earth materials, is generally considered the best for mortar, although excellent sand is often obtained from river-beds. Sea-sand contains alkaline salts which attract and retain moisture and cause efflorescence when used in brickwork, unless thoroughly washed.

Both sea- and river-sand have more or less rounded grains, to which lime or cement will not adhere as well as to sharp, angular grains. Both are extensively used, however, for lack of better materials.

The use of sand in mortar is to prevent excessive shrinkage, and to save the cost of lime or cement. Sand when used in the proportion of 1 to 2 strengthens lime mortar, but any addition of sand to *cement* weakens it.

Screening.—Sand for mortar must ordinarily be screened.

Sand for brown mortar for plastering or common brickwork is ordinarily run through a No. 4 screen having 4×4 meshes to the inch. For sand finish and mortar for pressed brickwork either a No. 10 or No. 12 screen with 10×10 or 12×12 meshes to the inch is commonly used.

For rubble stonework the sand is not ordinarily screened, unless it contains much gravel, in which case it should be screened through a $\frac{3}{8}$-in. mesh.

Weight.—Dry sand weighs from 80 to 115 lbs. per cubic foot. The average weight of damp (not wet) sand is about 96 lbs. per cubic foot, or 2,600 lbs. per yard.

The voids for ordinary sand range from .3 to .5 of the volume, the average for screened sand suitable for mortar being .35. The more uneven the grains in size the smaller the percentage of the voids.

A one-horse load of sand contains about 22 cu. ft. Two-horse loads vary from $1\frac{1}{4}$ to 2 yds. The amount hauled per load in the larger cities is generally fixed by the Team Owners' Association. $1\frac{1}{4}$ yds. is a fair load, $1\frac{1}{2}$ yds. a good load, and 2 yds. a large load.

LATHING AND PLASTERING.

(For more complete description of materials and processes see Chapter XI, Building Construction and Superintendence, Part I.)

Wooden Laths should be well seasoned, free from sap, bark, and dead knots. Bark on laths is quite sure to stain the plaster. White pine is generally considered as the best wood for laths, although spruce and hemlock laths are much used. Hard pine is not a good material, as it contains too much pitch.

The regular size of laths is $\frac{1}{4}$ in. $\times 1\frac{1}{2}$ in. $\times 4$ ft.; the width and thickness vary somewhat in different mills but the length is always the same. Laths are sold by the 1,000 in bunches containing 100 laths, $2.75 being about an average price.

For *Metal lathing* see pp. 825–831.

Plastering on laths is generally done in three coats.*

The first coat is called the *scratch* coat; the second, the *brown* coat, and the third, the *white* coat, *skim* coat, or finish.

On brick or stonework the scratch coat is generally omitted. For first-class work each coat should be permitted to dry thoroughly before the next coat is applied, and under no circumstances should the finish coat be applied before the brown coat is thoroughly dry.

"Drawn work" is where the brown coat is applied to the scratch coat from the same staging, immediately after the latter is applied. It is a little cheaper than "dry scratch," and much of this work is done in the Western States.

The scratch coat should always be made rich in lime, and should contain $1\frac{1}{4}$ bu. of hair (or an equivalent quantity of fibre) to each cask of lime, or 1 bu. of hair to 2 of lime.

A proportion of 1 part lime paste to 2 of sand will require 1 cask ($2\frac{1}{2}$ bu.) of lime to $5\frac{1}{2}$ bbls. of screened sand.

The brown coat should contain 1 cask ($2\frac{1}{2}$ bu.) of lime to 7 bbls. of screened sand, and 1 bu. of hair to 5 of lime.

Very little plaster is mixed by measure, however, the usual custom being to mix as much sand with the slaked lime as the mortar-mixer thinks it will stand and give satisfaction, the tendency being always to make the lime go as far as possible.

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* In the Eastern States, dwellings of moderate cost are generally plastered with two-coat work, the first or scratch coat being brought nearly to the grounds, and carefully straightened to receive the skim coat.

The third or finishing coat is designated by various terms, such as skim coat, white coat, putty coat, sand-finish, etc. The skim coat as used in the Eastern States is generally composed of lime putty and washed beach-sand in equal proportions. *Sand-finish,* which has a rough surface resembling coarse sandpaper, is mixed in the same way, only that coarser sand and more of it is used, and it is finished with a wooden or cork-faced float.

White coating or hard finish generally means a composition of lime putty and plaster of Paris, to which marble-dust is sometimes added. Plaster of Paris and marble-dust when used should not be mixed with the lime putty until a few moments before using, and no more should be prepared at one time than can be used up at once, as it soon "sets," after which it should not be used. The skim coat or hard finish should be finished with a steel trowel and wet brush. The more the work is trowelled the harder it becomes.

A superior hard finish is obtained by mixing 4 parts Best's Keene's cement to 1 part lime putty.

To make sure that the lime is well slaked, it is customary to require that the mortar for plastering shall be mixed at least seven days before it is used.

Hair such as is used by plasterers is obtained from the hides of cattle, and after being washed and dried is put up in paper bags, each bag being supposed to contain one bushel of hair when beaten up. Each package is supposed to weigh from seven to eight pounds, but the weight often falls short.

Asbestos and manilla fibre are both used in place of hair; they are cleaner than hair and are said to be less injured by the lime.

It is much better to add the hair to the lime paste *after it is cold* and before mixing in the sand, as hot lime, and the steam caused by the slaking, burn or rot the hair so as to greatly weaken it. The common practice is to put the hair in the mortar box, then run off the hot lime as soon as it is slaked, and then to throw in the sand and mix the whole together, when it is thrown out of the box into a pile and a new batch mixed up.

Machine-made Mortar.—In several of the larger cities plants have been equipped for the mixing of mortar by machinery. Machine-mixed mortar should be much better than the ordinary hand-mixed mortar, for the reason that time can be given

for the lime to slake, the lime and sand can be accurately measured, and the hair and lime are not mixed with the lime until just before delivery. The mixing may also be more thoroughly and evenly done by machinery than is possible by hand.

Improved Wall Plasters.—Owing to the difficulty of obtaining an economical and satisfactory quality of walls and ceilings by the use of the ordinary hand-mixed lime mortar, other and more reliable plastering materials have been invented, and are now being extensively employed, especially on the largest and most costly structures, and are giving general satisfaction.

Among the best-known of these improved plasters are the Acme, Agatite, and Royal cement plasters, Adamant, Windsor cement dry plaster, Rock wall plaster, and Best's Keene's cement. The first three are natural products found in certain parts of Kansas and Texas and simply calcined. Many other brands of these cement plasters are made in the Western States to supply the local markets. The other four plasters named above are composed principally of plaster of Paris with certain chemicals added. All appear to produce about the same results. The Windsor dry plaster, Adamant, and Rock plaster are mixed with the proper proportion of sand by the manufacturers, and only require being "wet up" before using. All of these materials are sold by weight. They should be used strictly in accordance with the directions furnished by the manufacturers.

Among the advantages gained by the use of these plasters are uniformity in strength and quality, extra hardness and toughness, freedom from pitting, saving in time required in making and drying the plaster, minimum danger from frost, less weight and moisture in the building, and greater resistance to the action of fire and water.

Measuring Plasterers' Work.—Lathing is always figured by the square yard and is generally included with the plastering, although in small country towns the carpenter often puts on the laths.

Plastering on plain surfaces, as walls and ceilings, is always measured by the square yard, whether it be one, two, or three-coat work, or lime or hard plaster.

In regard to deducting for openings, custom varies somewhat in different portions of the country and also with different

contractors. Some plasterers allow one half the area of openings for ordinary doors and windows, while others make no allowance for openings less than 7 sq. yds.

Returns of chimney-breasts, pilasters, and all strips less than 12 ins. in width should be measured as 12 ins. wide. Closets, soffits of stairs, etc., are generally figured at a higher rate than plain walls or ceilings, as it is not as easy to get at them. For circular or elliptical work, domes, or groined ceilings, an additional price is also made. If the plastering cannot be done from trestles an additional charge must be made for staging.

Stucco cornices and moulded work are generally measured by the superficial foot, measuring on the profile of the moulding. When less than 12 ins. in girth they are usually rated as 1 ft. For each internal angle 1 lin. ft. should be added, and for external angles, 2 ft.

For cornices on circular or elliptical work an additional price should be charged.

Enriched mouldings are generally figured by the lineal foot, the price depending upon the design and size of the mould.

Whenever plastering is done by measurement the contract should definitely state whether or not openings are to be deducted, and a special price should be made for the stucco-work, based on the full-size details.

Quantities of Materials Required for Lathing and Plastering.

To cover 100 sq. yds. requires from 1,400 to 1,500 laths, or say 1,450 for an average job, and 10 lbs. of 3d. nails.

Three-coat plastering on wood laths, plaster-of-Paris finish, will require from 8 to 10 bu. of lime, 1½ yds. of sand, 2 bu. of hair, and 100 lbs. of plaster of Paris per 100 sq. yds.

If finish coat is omitted, deduct 2 bu. of lime, and all of the plaster of Paris.

If sand-finished, omit the plaster of Paris and add ½ yd. of sand.

Two coats on brick or stone walls (brown coat and finishing coat) will require 6 to 8 bu. of lime, 1½ yds. of sand, and 100 lbs. of plaster of Paris, to 100 sq. yds.

Using Best's Keene's cement for brown mortar and *Keene's finish* on expanded metal lath will require, for brown mortar,

550 lbs. cement, 5½ bu. lime, 2 yds. sand, 2 bu. hair; for the finish, 300 lbs. cement and 1 bu. of lime per 100 yds.

Hard plasters on expanded metal lath, plaster-of-Paris finish, require for brown mortar 2,000 lbs. plaster and 2 yds. sand; for the finish, 1 bu. lime and 100 lbs. plaster of Paris per 100 yds.

Cost.—The standard price for putting on wood laths (labor only) in Denver (1904) is 3¾ cts. per yard. For expanded or sheet-metal laths on wood studding, 5 cts.; on steel studding, wired, 8 cts.

The cost of putting three coats on laths, plaster-of-Paris finish (labor only), runs about 15 cts. per yard for drawn work and 16 cts. for dry scratch.

With sand finish the cost is about the same as for white finish.

These figures are based on plasterers' wages at 62½ cts. per hour, and 37½ cts. per hour for hod-carriers and mortar-mixers.

The following table gives the average cost of different kinds of plastering in Denver in 1904, based on Missouri lime at 40 cts. per bushel, sand at 75 cts. per load of 1¼ yds., hair at 40 cts. per bushel, and plaster of Paris at 50 cts. per 100 lbs., and wages as given above.

Scratch and brown coat (lime) on wood laths	25 cts. per yd.
3 coats (lime) on wood laths, plaster-of-Paris finish	30 " " "
3 coats (lime) on wood laths, sand finish	30 " " "
Brown coat and finish on brick walls	23 " " "
For hard wall plaster instead of lime, add	3 " " "
3 coats (lime), plaster-of-Paris finish, metal lath on wood studding	65 " " "
3 coats (lime) plaster-of-Paris finish, metal lath on steel studding	68 " " "
For Keene's cement finish, add	10 " " "
For blocking in imitation of tile, add	50 " " "
2 coats hard wall plaster, plaster-of-Paris finish, metal lath, wood studding	70 " " "
2 coats hard wall plaster, plaster-of-Paris finish, metal lath on steel studs	73 " " "
For Keene's cement finish, add	10 " " "
Portland cement, brown coat, finished with Keene's cement blocked in imitation of tile, 3″ × 6″	$2.80 per yd.
For running base, 9″ high, in Best's Keene's cement	10 cts. per ft.

For running plain mouldings in plaster of Paris, 3 to 5 cts. per inch of girth.

For finishing shafts of columns, 16 to 24 ins. diam., 12 to 14 ft. high, $3 per column (labor only).

These prices are believed to be pretty near an average for the entire country. In some localities prices for materials or labor are less, in others higher.

Staff is a composition of plaster of Paris and hemp fibre, cast in moulds, and nailed or wired in place. All of the buildings of the Columbian Exposition at Chicago (1893) were covered with this material and all of the temporary buildings of the St. Louis Exposition of 1904.*

It is not sufficiently durable for permanent work unless kept well painted.

The cost of "staff" as used on the buildings at Chicago in 1893 varied from $2 to $2.25 per square yard.

DATA ON LUMBER AND CARPENTERS' WORK.†

Framing Lumber may commonly be purchased in any of the following sizes, except that common pine, spruce, and hemlock cannot usually be obtained in larger sizes than 12×12 ins.

2 ×4	3×6	4×12	8×12
2 ×6	3×8	4×14	8×14
2 ×8	3×10	6×6	10×10
2 ×10	3×12	6×8	10×12
2 ×12	3×14	6×10	10×14
2 ×14	3×16	6×12	10×16
2 ×16	4× 4	6×14	12×12
2½×12	4× 6	6×16	12×14
2½×14	4× 8	8× 8	12×16
2½×16	4×10	8×10	14×14
			14×16

In some of the New England mills, the following sizes are also sawn: 2×3, 2×5, 2×7, 2×9, 3×4, and 3×5.

These sizes are not commonly carried in stock, and in most localities would have to be obtained by ripping larger sizes.

* A description of the process of manufacture is given in Part I, Building Construction and Superintendence, p. 347.

† A comprehensive booklet giving the rules for the grading and classification of yellow-pine lumber and dressed stock may be obtained from the Southern Lumber Manufacturers Association, Equitable Building, St. Louis, Mo.

Most of the Southern yellow pine, Oregon pine, or Washington fir is *shipped surfaced one side and edge*—the actual dimensions being from $\frac{1}{4}$ in. to $\frac{3}{8}$ in. scant of the nominal dimensions, and sometimes $\frac{1}{2}$ in. When framing lumber is required to be full to dimensions it should be ordered "in the rough," and a special contract made on that understanding.

Length.—All timber is cut and sold in even lengths, as 10, 12, 14, and 16 ft. Odd and fractional lengths are counted as the next higher even length; consequently it is economical to plan buildings so that timbers of even lengths may be used without waste.

Measurement of Rough Lumber.—All rough lumber is sold by the foot, *board measure*, one foot being the equivalent of a board one foot wide, one foot long, and one inch thick.

To compute the board measure in any board, plank, or timber, divide the nominal sectional area, in inches, by 12, and multiply by the length in feet. Thus the number of "feet" in a 2×4 scantling 8 ft. long $=\frac{2\times4}{12}\times8=5\frac{1}{3}$ ft. b. m. A 10-inch board, 12 ft. long, contains $\frac{1\times10}{12}\times12=10$ ft. b. m.

Extensive tables are published showing the feet, board measure, in almost any commercial size of timber. The following table, however, although compact, will enable one to readily estimate the number of "feet" in any of the standard sizes of boards, planks, or timbers.

To use this table, multiply together, mentally, the dimensions of the cross-section, and then in the column having a heading equal to this product, and opposite the given length will be found the feet, board measure. Thus, for a 3×4, 2×6, or 1×12, look in column headed 12; for a 2×12, 4×6, or 3×8, look in column headed 24.

For lengths not given in the table, take either twice the length and divide by 2, or one half the length and multiply by 2.

Where timbers of the same size abut end to end, it economizes labor in reducing to board measure to take the full length; for this reason the lengths in the table are carried beyond that for a single stick.

TABLE OF BOARD MEASURE.

For explanation, see p. 1395.

Length in Feet.	Sectional Area in Square Inches.															
	4		6	8		10		12	14		16		18	20	
	ft.	ins.	ft.*	ft.	ins.	ft.	ins.	ft.*	ft.	ins.	ft.	ins.	ft.*	ft.	ins.
6	2	0	3	4	0	5	0	6	7	0	8	0	9	10	0
8	2	8	4	5	4	6	8	8	9	4	10	8	12	13	4
10	3	4	5	6	8	8	4	10	11	8	13	4	15	16	8
12	4	0	6	8	0	10	0	12	14	0	16	0	18	20	0
14	4	8	7	9	4	11	8	14	16	4	18	8	21	23	4
16	5	4	8	10	8	13	4	16	18	8	21	4	24	26	8
18	6	0	9	12	0	15	0	18	21	0	24	0	27	30	0
20	6	8	10	13	4	16	8	20	23	4	26	8	30	33	4
22	7	4	11	14	8	18	4	22	25	8	29	4	33	36	8
24	8	0	12	16	0	20	0	24	28	0	32	0	36	40	0
26	8	8	13	17	4	21	8	26	30	4	34	8	39	43	4
28	9	4	14	18	8	23	4	28	32	8	37	4	42	46	8
30	10	0	15	20	0	25	0	30	35	0	40	0	45	50	0
32	10	8	16	21	4	26	8	32	37	4	42	8	48	53	4
34	11	4	17	22	8	18	4	34	39	8	45	4	51	56	8
36	12	0	18	24	0	30	0	36	42	0	48	0	54	60	0
38	12	8	19	25	4	31	8	38	44	4	50	8	57	63	4
40	13	4	20	26	8	33	4	40	46	8	53	4	60	66	8
42	14	0	21	28	0	35	0	42	49	0	56	0	63	70	0

Length in Feet.	Sectional Area in Square Inches.													
	24	28		30	32		35		36	40		42	48
	ft.*	ft.	ins.	ft.*	ft.	ins.	ft.	ins.	ft.*	ft.	ins.	ft.*	ft.*
6	12	14	0	15	16	0	17	6	18	20	0	21	24
8	16	18	8	20	21	4	23	4	24	26	8	28	32
10	20	23	4	25	26	8	29	2	30	33	4	35	40
12	24	28	0	30	32	0	35	0	36	40	0	42	48
14	28	32	8	35	37	4	40	10	42	46	8	49	56
16	32	37	4	40	42	8	46	8	48	53	4	56	64
18	36	42	0	45	48	0	52	6	54	60	0	63	72
20	40	46	8	50	53	4	58	4	60	66	8	70	80
22	44	51	4	55	58	8	64	2	66	73	4	77	88
24	48	56	0	60	64	0	70	0	72	80	0	84	96
26	52	60	8	65	69	4	75	10	78	86	8	91	104
28	56	65	4	70	74	8	81	8	84	93	4	98	112
30	60	70	0	75	80	0	87	6	90	100	0	105	120
32	64	74	8	80	85	4	93	4	96	106	8	112	128
34	68	79	4	85	90	8	99	2	102	113	4	119	136
36	72	84	0	90	96	0	105	0	108	120	0	126	144
38	76	88	8	95	101	4	110	10	114	126	8	133	152
40	80	93	4	100	106	8	116	8	120	133	4	140	160
42	84	98	0	105	112	0	122	6	126	140	0	147	168

* The measurements in these columns come out in even feet.

TABLE OF BOARD MEASURE.—*Continued.*

For explanation, see p. 1395.

Length in Feet.	Sectional Area in Square Inches.													
	56		60	64		72	80		84	96	100		112	
	ft.	ins.	ft.*	ft.	ins.	ft.*	ft.	ins.	ft.*	ft.*	ft.	ins.	ft.	ins.
4	18	8	20	21	4	24	26	8	28	32	33	4	37	4
6	28	0	30	32	0	36	40	0	42	48	50	0	56	0
8	37	4	40	42	8	48	53	4	56	64	66	8	74	8
10	46	8	50	53	4	60	66	8	70	80	83	4	93	4
12	56	0	60	64	0	72	80	0	84	96	100	0	112	0
14	65	4	70	74	8	84	93	4	98	112	116	8	130	8
16	74	8	80	85	4	96	106	3	112	128	133	4	149	4
18	84	0	90	96	0	108	120	0	126	144	150	0	168	0
20	93	4	100	106	8	120	133	4	140	160	166	8	186	8
22	102	8	110	117	4	132	146	8	154	176	183	4	205	4
24	112	0	120	128	0	144	160	0	168	192	200	0	224	0
26	121	4	130	138	8	156	173	4	182	208	216	8	242	8
28	130	8	140	149	4	168	186	8	196	224	233	4	261	4
30	140	0	150	160	0	180	200	0	210	240	250	0	280	0
32	149	4	160	170	8	192	213	4	224	256	266	8	298	8
34	158	8	170	181	4	204	226	8	238	272	283	4	317	4
36	168	0	180	192	0	216	240	0	252	288	300	0	336	0
38	177	4	190	202	8	228	253	4	266	304	316	8	354	8
40	186	8	200	213	4	240	266	8	280	320	333	4	373	4
42	196	0	210	224	0	252	280	0	294	336	350	0	392	0
44	205	4	220	234	8	264	293	4	308	352	366	8	410	8
46	214	8	230	245	4	276	306	8	322	368	383	4	429	4
48	224	0	240	256	0	288	320	0	336	384	400	0	448	0
50	233	4	250	266	8	300	333	4	350	400	416	8	466	8
52	242	8	260	277	4	312	346	8	364	416	433	4	485	4
54	252	0	270	288	0	324	360	0	378	432	450	0	504	0
56	261	4	280	298	8	336	373	4	392	448	466	8	522	8
58	270	8	290	309	4	348	386	8	406	464	483	4	541	4
60	280	0	300	320	0	360	400	0	420	480	500	0	560	0
62	289	4	310	330	8	372	413	4	434	496	516	8	578	8
64	298	8	320	341	4	384	426	8	448	512	533	4	597	4
66	308	0	330	352	0	396	440	0	462	528	550	0	616	0
68	317	4	340	362	8	408	453	4	476	544	566	8	634	8
70	326	8	350	373	4	420	466	8	490	560	583	4	653	4
72	336	0	360	384	0	432	480	0	504	576	600	0	672	0
74	345	4	370	394	8	444	493	4	518	592	616	8	690	8
76	354	8	380	405	4	456	506	8	532	608	633	4	709	4
78	364	0	390	416	0	468	520	0	546	624	650	0	728	0
80	373	4	400	426	8	480	533	4	560	640	666	8	746	8
82	382	8	410	437	4	492	546	8	574	656	683	4	765	4
84	392	0	420	448	0	504	560	0	588	672	700	0	784	0

* The measurements in these columns come out in even feet.

TABLE OF BOARD MEASURE.—*Continued.*

For explanation, see p. 1395.

Length in Feet.	Size and Sectional Area in Inches.											
	120 10×12	140 10×14		144 12×12	160 10×16		168 12×14	192 12×16	196 14×14		224 14×16	
	ft.*	ft.	ins.	ft.*	ft.	ins.	ft.*	ft.*	ft.	ins.	ft.	ins.
4	40	46	8	48	53	4	56	64	65	4	74	8
6	60	70	0	72	80	0	84	96	98	0	112	0
8	80	93	4	96	106	8	112	128	130	8	149	4
10	100	116	8	120	133	4	140	160	163	4	186	8
12	120	140	0	144	160	0	168	192	196	0	224	0
14	140	163	4	168	186	8	196	224	228	8	261	4
16	160	186	8	192	213	4	224	256	261	4	298	8
18	180	210	0	216	240	0	252	288	294	0	336	0
20	200	233	4	240	266	8	280	320	326	8	373	4
22	220	256	8	264	293	4	308	352	359	4	410	8
24	240	280	0	288	320	0	336	384	392	0	448	0
26	260	303	4	312	346	8	364	416	424	8	485	4
28	280	326	8	336	373	4	392	448	457	4	522	8
30	300	350	0	360	400	0	420	480	490	0	560	0
32	320	373	4	384	426	8	448	512	522	8	597	4
34	340	396	8	408	453	4	476	544	555	4	634	8
36	360	420	0	432	480	0	504	576	588	0	672	0
38	380	443	4	456	506	8	532	608	620	8	709	4
40	400	466	8	480	533	4	560	640	653	4	746	8
42	420	490	0	504	560	0	588	672	686	0	784	0
44	440	513	4	528	586	8	616	704	718	8	821	4
46	460	536	8	552	613	4	644	736	751	4	858	8
48	480	560	0	576	640	0	672	768	784	0	896	0
50	500	583	4	600	666	8	700	800	816	8	933	4
52	520	606	8	624	693	4	728	832	849	4	970	8
54	540	630	0	648	720	0	756	864	882	0	1,008	0
56	560	653	4	672	746	8	784	896	914	8	1,045	4
58	580	676	8	696	773	4	812	928	947	4	1,082	8
60	600	700	0	720	800	0	840	960	980	0	1,120	0
62	620	723	4	744	826	8	868	992	1,012	8	1,157	4
64	640	746	8	768	853	4	896	1,024	1,045	4	1,194	8
66	660	770	0	792	880	0	924	1,056	1,078	0	1,232	0
68	680	793	4	816	906	8	952	1,088	1,110	8	1,269	4
70	700	816	8	840	933	4	980	1,120	1,143	4	1,306	8
72	720	840	0	864	960	0	1,008	1,152	1,176	0	1,344	0
74	740	863	4	888	986	8	1,036	1,184	1,208	8	1,381	4
76	760	886	8	912	1,013	4	1,064	1,216	1,241	4	1,418	8
78	780	910	0	936	1,040	0	1,092	1,248	1,274	0	1,456	0
80	800	933	4	960	1,066	8	1,120	1,280	1,306	8	1,493	4
82	820	956	8	984	1,093	4	1,148	1,312	1,339	4	1,530	8
84	840	980	0	1,008	1,120	0	1,176	1,344	1,372	0	1,568	0

* The measurements in these columns come out in even feet.

Measurement of Finishing Lumber, Flooring, Ceiling, etc.—Most, if not all, lumber for finishing is regular, sawn in thicknesses of 1 in., $1\frac{1}{4}$ in., $1\frac{1}{2}$ in., and 2 ins., and in some woods, such as white pine and poplar, is sawn $2\frac{1}{2}$ ins. and 3 ins. thick.

When surfaced both sides, the thickness is reduced to $\frac{13}{16}$, $1\frac{1}{16}$, $1\frac{5}{16}$, $1\frac{3}{4}$, $2\frac{1}{4}$, and $2\frac{11}{16}$ ins.

All dressed stock is measured and sold "strip count," *i.e.*, full size of rough material necessarily used in its manufacture. Thus $1\frac{1}{16}$-in. boards are measured as though $1\frac{1}{4}$ in. thick. The number of feet, board measure, for $1\frac{1}{4}$-in. stock ($1\frac{1}{16}$ finished) is $1\frac{1}{4}$ times that in a 1-in. board, and in the same way for $1\frac{1}{2}$-in. and $2\frac{1}{2}$-in. stock. $1\frac{3}{4}$-in. plank is always measured 2 ins. thick, and $2\frac{1}{4}$-in. stock, $2\frac{1}{2}$ ins. thick. Boards less than 1 in. thick are measured the same as inch boards, but for $\frac{3}{8}$- and $\frac{5}{8}$-in. stock a reduced price is generally made.

*Matched Flooring.**—The standard sizes are 1×3, 1×4, and 1×6, or $1\frac{1}{4}\times3$, $1\frac{1}{4}\times4$, and $1\frac{1}{4}\times6$. The thickness of 1-in. flooring should be $\frac{13}{16}$ in., and of $1\frac{1}{4}$-in. flooring, $1\frac{3}{32}$ in.; 3-in. flooring should show $2\frac{1}{4}$ ins. on face, after it is laid, 4-in., $3\frac{1}{4}$ ins., and 6-in., $5\frac{1}{4}$ ins.

Matched maple flooring is made in 2-in., $2\frac{1}{4}$-in., and $3\frac{1}{4}$-in. face, and in thicknesses of $\frac{7}{8}$, $1\frac{1}{8}$, and $1\frac{3}{8}$ ins.. There are three grades: Clear, No. 1, and Factory.

Ceiling (matched and beaded boards) is regularly stuck in the same widths as flooring. The standard (nominal) thicknesses of yellow-pine ceiling are $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{3}{4}$ in., the actual thickness of each being $\frac{1}{16}$ in. less. The $\frac{3}{8}$-in. ceiling is dressed one side only, the other thicknesses both sides.

Yellow-pine Drop Siding, all patterns, measures $\frac{3}{4}$ in.$\times5\frac{1}{2}$ ins. over all, and usually shows about 5-in. face.

Bevel Siding is resawed on a bevel from stock $\frac{13}{16}$ in.$\times5\frac{1}{2}$ ins. after surfacing.

The *New England Clapboards* are 4 ft. long, 6 ins. wide, $\frac{1}{2}$ in. thick at the butt, and about $\frac{1}{8}$ in. thick at the other edge. They are put up in bunches and sold by the thousand.

Rules for Estimating Quantities of Sheathing, Flooring, etc.—For common sheathing laid horizontally on a wall or roof without openings, add one tenth to the actual superficial area to allow for waste. On the walls of dwellings,

* Everywhere except in New England "flooring" is always understood to be tongued and grooved.

figure the walls as though without openings and allow nothing for waste. If sheathing is laid diagonally, add one sixth to the actual superficial area.

For tight sheathing laid horizontally, add one fifth for 6-in. boards, one seventh for 8-in. boards, and one ninth for 10-in. boards. If laid diagonally add one fourth for 6-in. boards, one-sixth for 8-in. boards, and one eighth for 10-in. boards.

For 3-in. matched flooring add one half to the actual superficial area to be covered.

For 4-in. flooring add one third and for 6-in. flooring add one fifth. Ceiling is measured the same as flooring.

For drop siding, add one fifth to the superficial area.

For lap siding laid 4 ins. to the weather, add one half to the actual superficial area; if $4\frac{1}{2}$ ins. to the weather, add one third.

Cost of Carpenters' Work.—There are so many items and conditions which enter into the cost of carpenters' work, and the cost varies so widely with the locality, that it is quite impossible to give figures which are of general practical value, although several books have been published on estimating carpenters' work. The best of these that the author has seen is "Estimating Frame and Brick Houses," by Fred T. Hodgson.

The following figures of the cost (for labor and nails) of framing and putting on sheathing and siding and laying flooring are probably a fair average, with carpenters' wages at \$3 a day of eight hours ($37\frac{1}{2}$ cts. per hour). The cost of framing is almost always figured at a certain price per thousand feet of lumber, board measure. The cost of laying flooring-sheathing, etc., is always figured by the square of 100 sq. ft. ($10' \times 10'$).

For setting up studding and framing walls of wooden dwellings,	\$10.00	per M.
For framing and setting floor joists, 2×8 to 2×12....	\$9 to \$10	" "
Framing and setting heavy joists and girders, 6×12 to 10×14,	\$8.50	" "
Framing gable roofs and setting in place..................	\$10.00	" "
Framing hip roofs and setting in place.................	\$11 to \$12	" "
For putting in bridging, after it is cut, per 100 lin. ft. in the row,	\$1.25	
For covering the sides or roofs of wooden buildings with dressed sheathing, laid horizontally	0.60	per square
If laid diagonally..................................	0.75	" "
The cost of labor and nails for laying 6″ flooring, blind nailed to every joist without dressing after laying is about..................................	2.00	" "
For 4″ flooring, not dressed, allow....................	2.25	" "
For 3″ flooring, not dressed, allow....................	2.50	" "
For 3″ hard-pine flooring, hand smoothed or traversed....	3.75	" "
For 3″ red-oak flooring, hand smoothed or traversed......	6.00	" "
For 3″ white-oak flooring, hand smoothed or traversed....	8.00	" "
For 3″ maple flooring, hand smoothed or traversed..	\$10 to \$12	" "

BUILDING PAPERS, FELTS, QUILTS, ETC.

There is a great variety of papers and felts manufactured for use on buildings. They may be broadly classified as follows:

Rosin-sized Building Papers.*—These are about the cheapest grades of building paper; they are not water-proof, and should not be used on roofs, or on walls in damp climates. In dry places they protect from dust, draughts, and to some extent from heat and cold. They are generally either a dull red or gray in color, have a hard smooth surface, and are clean to handle. Always put up in rolls 36 ins. wide and usually containing 500 sq. ft. Weight varies from 18 to 40 lbs. to the roll of 500 sq. ft.; cost, from 50 cts. to $1.50 per roll.†

Water-proof Papers.—*Neponset Black Sheathing* is water- and air-proof, odorless and clean to handle. An excellent paper under siding, shingles, slate, or tin. Rolls 36 ins. wide, containing 250 and 500 sq. ft.; cost, about $2.00 per roll of 500 sq. ft.

Neponset Red Rope Sheathing and Roofing.—Made of rope stock; has great strength and flexibility, absolutely water-proof and air-tight. One of the best sheathing papers. Makes a good cheap roofing for sheds, poultry-houses, etc. Rolls 36 ins. wide, containing 250 and 500 sq. ft. Cost, about $5.00 per 500 sq. ft.

Parchment Water-proof Sheathing.—Semi-transparent, smooth surface, odorless, water-, air-, and vermin-proof. Adapted for general sheathing purposes and for use in concrete construction. 1-ply, 25 lbs. to 900 sq. ft.; 2-ply, 25 lbs. to 500 sq. ft.; 3-ply, 25 lbs. to 275 sq. ft. All 36 ins. wide.

P. & B. Building Paper.—Thoroughly coated with P. & B. compound (principally paraffine), is water-, acid-, alkali-, and gas-proof; claimed not to decay. An excellent sheathing paper. Black and glossy, but not sticky. Rolls 26 ins. wide, containing 1000 sq. ft. Made 1-ply (very thin), 30 lbs.; 2-ply, 40 lbs.; 3-ply, 65 lbs.; 4-ply, 80 lbs. Cost, $3.00, $4.50, $6.00, and $8.00, respectively.

Dry Felts.—Common felts are composed of waste vegetable fibres cemented together with rosin. Better grades are made from wool stock. Felts are made in many different thicknesses, and in 32-in. and 36-in. widths. They should be specified by *weight* unless a particular brand is specified. Common dry felt weighs from 4½ to 5 lbs. per 100 sq. ft.

Barrett's Eureka Brand.—All-wool stock, 32 ins. wide, and weighs 1 lb. to the square yard.

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* The terms "building" and "sheathing" are indiscriminately applied to all kinds of papers used in connection with building construction. In the trade, however, the term "building paper" is confined to the rosin-sized and cheaper grades of paper, while the heavier and better grades are classed as sheathing papers.

† All prices are approximate; they vary with locality and condition of the market.

Barrett's Excelsior Brand.—All-wool stock, 32 ins. wide, and weighs 1½ lbs. to the square yard. This is a very heavy felt.

A dry wool felt weighing 1 lb. to the square yard will be about ⅛ in. thick. Such felts are used principally for deadening between floors and as carpet lining. Commonly sold by the pound, 2¼ cts. a pound being perhaps an average price.

Saturated Felts.—Common roofing felts are made by saturating common dry felt with coal-tar pitch. Roofing felts are commonly made in weights of 12, 15, and 20 lbs. to the 100 sq. ft. Nothing lighter than 12 lbs. should be used for roofing. Usually sold by weight. Average price, 1½ cts. a pound.

Asphalt felts are commonly made in the same weights.

Dry Saturated Tarred Felts are specially run through a tier of calenders to give a hard, uniform surface and contain a minimum amount of coal-tar. Are especially adapted for slaters' use, as they will carry a chalk line and are easy to handle. Rolls 36 ins. wide contain 500 sq. ft. and weigh about 30 lbs. Cost, about 80 cts. per roll.

Asbestos Building Felts are usually made about 6, 10, 14, and 16 lbs. to the 100 sq. ft., although different manufacturers make different weights. Rolls 36 ins. wide. Sold by weight.

Insulating and Deadening Quilts.

Cabot's "Quilt" consists of a felted matting of eel-grass held in place between two layers of tough manila paper by "quilting." Also made with a covering of asbestos. Single-ply weighs 85 lbs. per bale of 500 sq. ft., width 36 ins. Double-ply weighs 125 lbs. per bale of 500 sq. ft., width 36 ins.

Keystone Hair Insulator.—A quilt with hair filling. Four brands, each packed in bales 3 ft. wide containing 500 sq. ft. *Acme*, plain paper both sides, weight per bale 60 lbs. *Neptune*, water-proof paper one side, plain paper other side, weight per bale 70 lbs. *Phœnix*, asbestos paper one side, plain paper other side, weight per bale 100 lbs. *Salamander*, asbestos paper both sides, weight per bale 130 lbs.

The Union Fibre Company's Mineral-wool Deafener is made of rock-fibre wool, quilted between sheets of rosin-sized, water-proof or fire-proof paper. Put up in rolls 36 ins. wide, ½ in. thick, and containing 125 sq. ft.

The Union Fibre Company's Flax-fibre Floor Deadener is made of degummed flax fibre, sewed between two thicknesses of rosin-sized paper. Put up in rolls 36 ins. wide, ¼ in. thick, and containing 200 sq. ft. Also furnished with water-proof or asbestos paper covering.

Cost of Building and Sheathing Papers in Place.

The following, although necessarily restricted to a few lines, will give a general idea of the cost of different kinds and grades of sheathing papers, the price given being a fair average for the material *applied* to an outside wall or roof:

	Price per 100 Square Feet.
Common tarred felts (15 lbs. per square)*	$0 30
Red rosin-sized sheathing, best grades	0.35
Manahan's parchment sheathing, single-ply	0.26
" " " double-ply	0.40
" ship-rigging tar sheathing, 2-ply	0.75
"Neponset" black (water-proof) building paper	0.45
" red rope roofing fabric	1.10
Sheathing papers with asphalt centre	$0.40 to 0.50
Johns' asbestos building felt, 10 lbs. per square	0.42
" " " " 14 lbs. per square	0.55
Cabot's sheathing quilt, single-ply	1.05
" " " double-ply	1.25
Sawyer's century sheathing quilt (felt coated one side with a water- and vermin-proof compound)	1.35

Paint and Varnish.

The solid ingredient of a paint is called the pigment, and is a fine powder, nearly all of which will pass through a brass wire sieve of 100 meshes to the linear inch; in fact, most pigments are much finer than that, and those formed as precipitates by chemical processes are so fine that there is no way to measure them.

The liquid part is called the vehicle; this is usually linseed oil, sometimes with the addition of a little turpentine or other volatile solvent; in the enamel paints it is varnish; in kalsomine and other cold-water paints it is a solution of glue, casein, albumen, or some similar cementing material. The cementing material is sometimes called the binder.

Ingredients of Oil Paint.—White lead and white zinc are the common white pigments, but there is also a white pigment called zinc-lead, which is made in a furnace from ores containing lead and zinc, and is of variable composition; and lithopone, a mixture of sulphide of zinc and sulphate of barium; this sometimes darkens by exposure to light, and is discolored if it comes in contact with anything containing lead. Yellow is commonly chromate of lead, or chrome-yellow; green is chrome-green, which is a mixture of chrome-yellow and prussian blue; blue is ultramarine, or sometimes prussian blue. The brilliant reds are coal-tar colors as a rule; the dull reds and browns are oxides of iron. Ochres are dull yellow. Carbon forms the base of all black paints, either as lampblack, dropblack (boneblack), or graphite.

Linseed oil is either raw or boiled. Raw oil is the oil in its

* A "square" is 100 sq, ft.

natural state as it is extracted from the seed; it should be settled and filtered perfectly clear; it is yellow or greenish yellow in color. Boiled oil is raw oil which has been heated to 400 or 500 F. with compounds (usually oxides) of lead and manganese; it is darker in color than raw oil, and dries more quickly. Raw oil exposed in a thin film to the air is converted in about five days into a tough leathery substance; boiled oil undergoes this change in ten to twenty-four hours.

Driers. — These are compounds of lead and manganese, dissolved in oil, and this solution thinned with turpentine or benzine. They act as carriers of oxygen between the air and the oil, and their addition to a paint makes it dry more rapidly. Some driers are also called japans. Not more than 10% by volume of any of these liquid driers should be added to oil. Excess of drier causes the paint to lack durability.

Priming Coat. — This is the first coat applied to the clean surface; a priming coat for wood is chiefly oil, and is usually equivalent to a gallon of ordinary paint thinned with a gallon of raw linseed oil. But paint is not thinned to make a priming coat for structural metal. In all woodwork nail-holes and other defects are filled with putty after the priming coat has been applied; but if the wood is resinous, knots and resinous places must be covered with shellac varnish before the priming coat is put on.

Outside Painting.—The priming coat having largely been absorbed by the wood, a second and third coat of paint are to be applied. The most common paint used on houses is white lead. This is usually sold as paste white lead, containing 10% of oil; 100 lbs. of this mixed with 5 gal. oil makes $6\frac{1}{2}$ gal. paint weighing about $21\frac{1}{4}$ lbs. per gal.; a gal. of this contains about 14 lbs. dry white lead. White zinc is also an important paint; $9\frac{1}{2}$ lbs. zinc and 5.7 lbs. oil make a gallon of paint.

The defect of white lead is that it "chalks," i.e., in time its surface becomes a dry powder, which rubs off; the defect of zinc is that it becomes hard and shows a tendency to peel off; the two are mixed with good results, $\frac{1}{4}$ to $\frac{1}{2}$ the mixture being zinc. Colored paints are made by adding colored pigments to zinc or lead; but dark colored paints often contain only iron oxides, ochres, etc., as pigments; these weigh 12 to 14 lbs. per gallon. In applying paint, allow a week between successive coats. In painting the outside of a house, paint the

trim first, then the body color can be laid neatly against it. It is desirable to brush it on with the grain of the wood.

A gallon of priming coat will cover about 300 sq. ft., and a gallon of paint used in subsequent coats about 600 sq. ft. Roof paints should contain a larger proportion of oil, and less drier or none. Three coats are desirable. Tin roofs and galvanized iron work should be thoroughly scrubbed and then dried before painting. Shingles on the wall of the house (and sometimes on the roof) are sometimes stained with creosote stain, which consists of a pigment suspended in creosote or some similar liquid. The creosote has some preservative effect.

Inside Painting. — Door and window frames should receive a priming coat of paint in the shop; if they are to be finished in varnish this paint will be applied to the back only. As has already been said, before any painting is done any resinous knots are to be varnished with shellac. All interior surfaces which are to be painted should be puttied after the priming coat; the putty should be applied with a wooden spatula, not a steel one, to avoid marring the surface. The paint for the second coat should contain as much turpentine as oil, i.e., its vehicle should be half oil and half turpentine. The effect of this is to make the paint dry with a dull surface, "flat" is the painter's term; not glossy; to this the next coat will adhere well. If the next is the final coat this may be an ordinary oil paint. When thoroughly dry the gloss may be removed by lightly rubbing it with pumice and water.

Enamel paint consists of pigment with varnish as a vehicle, and is harder and makes a finer finish than oil paint. It is also more expensive. It is usual to apply it over oil paint; in this case the last coat of oil paint should be lightly sandpapered when quite hard and dry, then put on a coat of enamel paint; when this is dry it should be sandpapered or rubbed with curled hair, and the final coat of enamel is then laid on; this may be rubbed in a like manner if a flat surface is desired, or it may be left with the gloss.

Varnish. — There are two principal kinds of varnish, spirit varnishes, of which shellac varnish is the most important, and which consists essentially of a resin dissolved in a volatile solvent, and oleo-resinous varnishes, in which the resinous ingredient is combined with linseed oil, and this compound is dissolved in turpentine or benzine. The oleo-resinous varnishes are commercially the more important, and are largely

used in interior finishing. A gallon of varnish covers 500 sq. ft. one coat. Surfaces to be varnished are treated in the following manner: If the wood is of the open grained sort, such as oak, chestnut or ash, it first receives a coat of paste filler. Liquid fillers are not desirable, as they form a poor base for subsequent work. A paste filler is really a sort of paint, the pigment being silex, or ground quartz, and the vehicle is a quick-drying varnish made thin with turpentine or benzine. This is rubbed strongly in on the grain of the wood with a short stiff brush, and as soon as it has set, usually within half an hour, it is rubbed off with a harsh cloth or a handful of excelsior, rubbing hard across the grain of the wood. If it is desired to stain the wood, the oil stain may be mixed with the filler; but if a close-grained wood is used, which needs no filler, the oil stain may be thinned to the desired color with turpentine or benzine and applied as a wash. In cleaning the filler out of mouldings, corners, etc., a suitably shaped stick may be used but not a steel implement. If any puttying is necessary it is next done. After two days the first coat of varnish is applied; after five days it should be rubbed with curled hair or fine sandpaper to remove the gloss, so that the next coat will adhere well; then one, two or three more coats of varnish, five days or more apart, each coat rubbed except the last, which may be rubbed or left with the natural gloss. Outside doors, window sills, jambs, inside blinds, and all surfaces exposed to the direct rays of the sun, are varnished with spar varnish and left glossy. If shellac varnish is used as the interior finish it is applied in the same way, but at least six coats should be applied. Floors which are to be varnished should be treated as has been described; but if they are to be waxed they should receive one or two coats of shellac varnish, then five or six coats of wax, at intervals of a week, each coat being well polished with a weighted floor-brush made for the purpose. Floor-wax is not beeswax, but is a compound wax made for the purpose. Shellac is a good floor-varnish; it discolors the wood less than any other varnish, and dries rapidly.

Plastered walls which must be painted are usually washed with a solution of soap and then with a solution of alum. When this is dry it is sponged off, then allowed to dry, then oiled, then painted. If the paint is applied to the fresh plaster the lime in the plaster will attack the paint.

Repainting.—The exterior woodwork of a house needs repainting once in five to ten years, according to climate and similar conditions, although if not done with proper material or sufficient care it will not last as long as this; the interior should, with good care, stand fifteen to twenty years, and then may not require complete renewal. Exterior paint sometimes loses its luster, while the body of the paint is still good, and in such a case it is enough to wash the surface and then give it a coat of oil; this will replace the oil which has superficially perished and will impart gloss and bring out color. If the paint is worn off so as to show the wood in places, or is peeling, it must be very carefully examined. In extreme cases it will be necessary to "burn off" the old paint; this is done with a painter's torch, a lamp which burns alcohol, naphtha or kerosene, and which furnishes a flaring blast of flame, which is directed against the painted surface just long enough to soften the paint which is at once removed, while still hot, with a scraper. The paint is not actually burned, only softened by the flame; but it may thus be removed completely. Houses covered with pitchy wood, like southern pine, sometimes require this treatment, and the next painting is found to be more lasting. In many cases it will be sufficient to thoroughly scrub the surface with a stiff steel wire brush. Interior surfaces may be cleaned (if the removal of the old paint and varnish is necessary) with varnish remover; this is a mixture of solvent liquids, which penetrate the old paint or varnish and softens it, when it may be removed with scrapers or brushes. It is less dangerous on account of fire than the burning-off method, but slower and more costly. It must not be forgotten that varnish remover is volatile and highly inflammable and must not be used in a room where there is a fire. It is especially suitable for cleaning out mouldings and all irregular surfaces, from which the varnish may then be removed with stiff brushes, if not convenient to use scrapers. It is especially desirable to clean floors occasionally in this way; but if the house has been varnished originally with a first-class varnish it may be necessary only to wash it thoroughly and then apply another coat of varnish. Smoke and dirt may often be removed perfectly from ceilings with the crumbs of fresh bread, where washing would not be desirable. A ten per cent solution of carbonate of soda (sal soda) in hot water may be used to remove old floor-wax.

Paint for Structural Steel. — Steel being usually more perishable than wood, as well as more expensive, and used for service where its strength is essential to the stability of the structure, its prevention from corrosion by painting is of much importance. It must first of all be recognized that the precaution always taken in painting wood to secure a clean surface for the paint, must not be omitted with steel. Mud and dirt which have got on it must be first removed; then it must be examined for rust, and any rusty spots must be cleaned thoroughly; loose scale may be removed by wire brushes, but thick and closely-adherent rust must be removed with steel scrapers, or with hammer and chisel if necessary.

No doubt the best way to clean steel is with the sand-blast, but it is not available for much architectural work; but much care must be taken to obtain a clean surface. On wood the priming coat sinks into the wood and forms a perfect bond between it and the succeeding coats; but with metal no such thing is possible; it is a case of simple adhesion, and to adhere well the surface must be clean. The paint for structural metal should be tough and elastic, and to as great a degree as possible it should be water-proof. Less than two coats should never be applied, and three are better. Paint is always thin on edges and angles, also on bolt and rivets heads; it is therefore good practice, after the first full coat, to apply a partial or striping coat, covering the angles and edges for at least an inch back from the edge, and covering all bolt and rivet heads. After this striping coat has become dry, the second full coat it applied; it may then be assumed that the whole surface has received two full coats. At least a week should elapse between coats.

In designing the steel-work, all cavities which may be filled with rain during erection should be properly drained; and during erection all small cavities should be filled with cement, and all contact surfaces should be thickly painted.

Kinds of Paint.—One of the most widely used paints for structural steel, especially as a first coat, is red lead; it adheres well, it dries, not like an ordinary paint by oxidation, but by combining directly with the oil, as a cement; it protects the steel from carbonic acid to a considerable degree, because the lead it contains is easily acted on chemically. This very fact, however, may make it less permanent, unless

it is protected by some other more neutral paint. In pure and reasonably dry air red lead sometimes shows much permanence, but in the air of cities and near the sea coast it is probably desirable to use it only for undercoat work. Red lead should be freshly prepared from day to day; 30 to 33 lbs. dry red lead to each gallon of oil is about right; the so-called "ready-mixed" red lead paints are less valuable.

Finely ground graphite in linseed oil is a favorite paint for metal; it flows well, is easily applied, less expensive than red lead, and if well made gives exellent results.

Graphite is sometimes mixed with lamp-black, probably with advantage. Boneblack is also an important ingredient of "carbon" paints. Formerly oxide of iron in linseed oil was used more than all other paints for this purpose; but while many engineers still like it, its use has very greatly diminished. Asphaltum has been used and is still used, as a varnish either alone or in combination, and some of these asphaltic preparations are fairly satisfactory. The fact is, that a really competent paint manufacturer can make a reasonably good paint out of any of these, and if the application is well and carefully done the results will be far in advance of what is commonly seen, and there are great differences in painters.

As regards the surface of structural steel covered by a gallon of paint, there is much difference of opinion among experts. Some say 300 to 400 sq. ft., others as high as 1000 or 1200. The truth is that any paint may be by a skilled workman brushed out into an exceedingly thin film; while ordinary usage gives one at least twice as thick. The most general opinion is that it is not wise to estimate more than 400 sq. ft. to the gallon, one coat. Varnish paints cover less than oil, but if well made they are more durable.

GLASS—KINDS AND PRICE LISTS.

Sheet Glass or Common Window Glass.—Common window glass is technically known as sheet or cylinder glass because it is first blown into the form of a cylinder, then cut longitudinally and flattened on a stone. Sheet glass can readily be distinguished from plate glass, even at a distance, because of its wavy appearance, which cannot be wholly avoided.

Grades and Qualities of Sheet Glass.—All common sheet glass, without regard to quality, is graded according to thickness, as "single strength" or "double strength." The double-strength glass is supposed to have a nearly uniform thickness of $\frac{1}{8}$ in., while the single strength may be as thin as $\frac{1}{16}$ in. The thickness of single-strength glass, however, is generally far from uniform.

Both single- and double-strength glass are sorted into three grades or qualities, the classification depending upon color, brilliancy, and flaws.

In the common American glass the best quality is designated as AA, the second as A, and the third as B. The AA quality is supposed to be as good glass as can be made by the cylinder process. As even this glass, however, is not entirely free from defects, it is very difficult for any one but an expert to tell exactly whether certain lights of glass are first or second quality. The A quality is most used. The B quality is only suitable for cellar windows, stables, factories, greenhouses, etc.

Sizes.—The regular stock sizes vary by inches from 6 to 16 ins., and above that by even inches up to 60 ins. in width and 70 ins. in height for double strength and 34×50 ins. for single strength.

Cost.—The price for sheet glass, as for all other clear glass, varies with the size, strength, and quality. It is determined by a schedule, or price-list, fixed from time to time by the glass companies, from which a very large discount is made, fluctuations in prices being regulated by the discount, which at present is about 90 and 20 off for St. Louis, the discount varying with the freight rate. The list on common glass is changed more frequently than that for plate glass; the present list having been adopted Oct. 1, 1893. The only way of ascertaining the price of a light of glass of a given size is by means of the price-list and discount.

The price per square foot increases rapidly as the size of the glass increases, so that it is much cheaper to divide a large window into eight or twelve lights than into two lights.

The table on page 1417 gives the present list price for single lights of the sizes most commonly used. The net price is obtained by deducting the discounts as illustrated by the example under Plate Glass.

The price by the box is about 15 per cent. less than for single lights.

Polished Plate Glass.—Plate glass is the highest grade of window glass, being cast in large sheets on a flat table and then polished, while the common sheet glass is blown. It is manufactured in sheets of various sizes, some as large as 12 ft. wide by from 15 to 16 ft. long. The average thickness is from $\frac{1}{4}$ to $\frac{5}{16}$ in.

The cost varies according to the size of the light. To ascertain this cost a regular price-list is used, which is subject to a large discount. This list is the standard list for the entire trade and is maintained from year to year, rarely changing, the present price-list having been established in 1894. The

LIST PRICE OF COMMON WINDOW GLASS IN 1904.

Prices are for single lights, A quality; single strength up to and including 15″×40″, double strength above.

Size in Inches.	Price per Light.	Size in Inches.	Price per Light.	Size in Inches.	Price per Light.	Size in Inches.	Price per Light.
6×8	$0.21	16×20	$2.28	24×32	$5.98	32×48	$14.15
7×9	0.27	16×24	2.76	24×34	6.65	32×60	19.55
8×10	0.35	16×28	3.56	24×36	6.65	32×72	33.26
8×12	0.42	16×30	3.80	24×40	8.05	36×36	11.79
9×12	0.46	16×32	4.07	24×44	9.20	36×40	14.15
9×15	0.59	16×36	4.49	24×48	11.79	36×44	14.15
10×12	0.52	16×40	5.44	24×60	14.44	36×48	18.05
10×14	0.59	16×48	7.16	24×72	23.00	36×60	30.67
10×16	0.72	18×24	3.35	26×28	5.98	36×72	37.38
10×18	0.81	18×30	4.07	26×30	6.65	36×84	48.59
12×14	0.75	18×32	4.38	26×32	6.65	40×40	14.15
12×15	0.81	18×36	5.31	26×34	8.05	40×48	19.20
12×16	0.85	18×40	5.98	26×36	8.05	40×60	30.67
12×18	0.95	18×48	8.05	28×30	6.65	40×72	41.40
12×24	1.38	18×60	10.31	28×32	8.05	40×84	53.77
12×30	1.83	20×22	3.56	28×34	8.05	44×44	21.12
12×32	1.93	20×24	3.80	28×36	9.20	44×48	28.68
14×14	0.88	20×26	4.07	28×40	9.20	44×60	36.59
14×16	1.01	20×28	4.38	28×44	11.79	44×72	53.45
14×18	1.12	20×30	4.75	28×48	14.15	48×48	33.74
14×20	1.24	20×32	5.31	28×60	19.20	48×56	36.59
14×24	1.57	20×36	5.98	28×72	23.00	48×60	41.12
14×28	1.93	20×40	6.65	30×30	8.05	48×72	53.45
14×30	2.15	20×44	8.05	30×32	9.20	50×50	33.74
14×32	2.29	20×48	8.05	30×34	9.20	50×60	41.12
14×36	2.61	20×60	12.03	30×36	9.20	50×72	59.15
14×40	2.90	22×24	4.07	30×40	10.74	54×60	53.45
15×16	1.08	22×28	4.75	30×44	11.79	54×72	64.84
15×18	1.20	22×30	5.31	30×48	14.15	56×60	53.45
15×20	1.44	22×32	5.84	30×60	19.20	56×72	64.84
15×24	1.73	22×36	6.65	30×72	33.26	60×60	53.45
15×30	2.29	22×40	8.05	32×32	9.20	60×62	59.15
15×32	2.44	22×48	9.20	32×34	9.20	60×64	59.15
15×36	2.90	24×26	4.75	32×36	10.74	60×66	64.84
15×40	3.33	24×28	5.31	32×40	11.79	60×68	64.84

fluctuations in the selling price are arranged by means of a discount which is the same for all sizes. At the present time (July 1904) the discount to builders is about 80 and 5 off at Denver. Nearer Pittsburgh the discount is greater on account of lower freight charges. This discount is liable to sudden changes. The price-list now in effect, slightly abridged, is given on pp. 1420–1423.

Examples of Figuring Cost.—What is the net cost of a light of plate glass 72×96 ins., the discount being 80 and 5 per cent? *Ans.* The list price, in table, is \$173; 80 per cent=\$138.40; subtracting from \$173 we have \$34.60; 5 per cent off from this leaves \$32.87, the net price.

Odd and fractional parts of inches are charged at the price of the next highest even inches. Thus 31×120½ ins. costs the same as 32×122 ins.

The average weight of plate glass is 3½ lbs. to the square foot.

Cost of Bending Plate and Window Glass.—Official scale, adopted March 1, 1900:

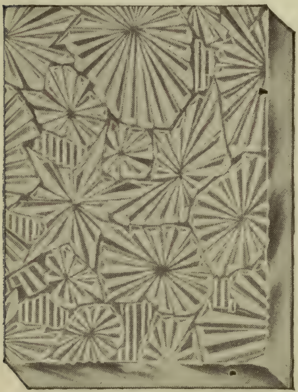
PLATE GLASS.

Plates where length and width are added, less than 76 inches.	$0.60	sq. ft.
" of 76 or more but less than 90 united inches	0.75	" "
" " 90 " " " " " 100 " "	1.00	" "
" " 100 " " " " " 110 " "	1.50	" "
" " 110 " " " " " 120 " "	2.00	" "
" " 120 " " " " " 140 " "	2.50	" "
" " 140 " " " " " 160 " "	3.00	" "
" " 160 " " " " " 180 " "	3.50	" "
" " 180 " " " " " 200 " "	4.00	" "
" " 200 " " " " " 210 " "	4.50	" "
" " 210 " " " " " 220 " "	5.00	" "
" " 220 " " " " " 230 " "	5.50	" "
" " 230 " " " " " 240 " "	6.00	" "

WINDOW GLASS.

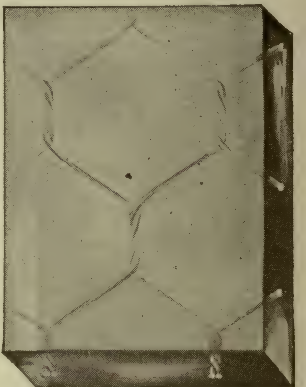
Lights of less than 60 united inches	$0.25	sq. ft.
" " 60 or more but less than 70 united inches	0.30	" "
" " 70 " " " " " 80 " "	0.40	" "
" " 80 " " " " " 90 " "	0.50	" "
" " 90 " " " " " 100 " "	0.60	" "
" " 100 " " " " " 110 " "	0.90	" "

Figured Rolled Glass.—The Mississippi Glass Company manufactures nine different patterns of figured rolled glass for use in doors, transoms, and windows where an obscure glass is desired, or for purely ornamental effects. The Maze and Ondoyant patterns are especially valuable either



FULL-SIZE DETAIL OF FIGURED ROLLED-GLASS, "FLORENTINE" PATTERN.

Other popular patterns specified are "Ondoyant," "Maze," "Venetian," "Syenite," "Oceanic," and "Figured No. 1," "Figured No. 2," and "Figured No. 3." Thicknesses ⅛ and 3/16 in.; widths, 30, 40, and 42 ins.; lengths about 100 ins.



FULL-SIZE DETAIL OF ROLLED WIRE GLASS, "ROUGH" or "HAMMERED" STYLE.

Other popular patterns specified are "Maze" and "Ribbed" and "Polished." ¼ in. thick is standard and is the only thickness "Polished." "Maze," "Ribbed," "Rough," or "Hammered" can be had ⅜ in. thick; widths up to 40 ins.; lengths up to 120 ins.

in outside windows or skylights. For diffusing the light see pp. 1300–1303. The Maze pattern may be had either with or without embedded wire. Ondoyant glass is made ⅛ in. thick and 30 ins. wide. The Maze and several other patterns of rolled glass are made ⅛ and 3/16 in. thick, and 42 ins. wide.

Maze wired glass is made in sheets ¼ and ⅜ in. thick, up to 40 ins. wide and 100 ins. long.

Figured glass, on account of its greater cleanliness and diffusing qualities, has almost entirely supplanted ground glass and, to a considerable extent, chipped glass.

Wire Glass is described on p. 765.

Prismatic Glass, for glazing windows, skylights, and sidewalk lights, is now manufactured in a large number of forms in both prisms and sheets, and by several companies, the more important of which are as follows:

American Luxfer Prism Co.	Chicago, Ill.
American Prismatic Light Co.	Philadelphia, Pa.
Cleveland Window Glass Co.	Cleveland, Ohio.
Daylight Glass Mfg. Co.	Philadelphia, Pa.
Daylight Prism Co.	Chicago, Ill.
New York Prism Co.	New York, N. Y.
Solar Prism Co.	Cleveland, Ohio.

The diffusing properties of several types are described on pp. 1300–1303.

Glass for Skylights.—Where skylights are glazed with clear or double thick glass, it may be used in lengths of from 16 to 30 ins. by a width of from 9 to 15 ins. A lap of at least an inch and a half is necessary for all joints. This is the cheapest mode of glazing. The best glass, however, for skylight purposes, next to prism or wire glass (see p. 1304), is fluted or rough plate glass. The following thicknesses are recommended as proportionate sizes:

12 inches by 48 inches is the extent for glass 3/16 inch thickness.
15 " " 60 " " " " " " ¼ " "
20 " " 100 " " " " " " ⅜ " "
94 " " 156 " " " " " " ½ " "

WEIGHT OF ROUGH GLASS PER SQUARE FOOT.

Thickness	⅛	3/16	¼	⅜	½	⅝	¾	1 in.
Weight	2	2½	3½	5	7	8½	10	12½ lbs.

The cost of skylights with galvanized-iron frame, glazed with 3/16-in. or ¼-in. ribbed glass, ranges from 40 to 60 cts. per square foot of area covered.

Cost of Rolled Glass.—In 1904, the different kinds of glass were quoted for small quantities in St. Louis about as follows:

3/16-in. ribbed skylight glass	8 cts. per sq. ft.
¼ " " " "	12 " " " "
Ribbed wire glass, ¼ in. thick	23 " " " "
Maze " " " " "	23 " " " "
Factory ribbed glass, ⅛ in. thick	9 " " " "
Ondoyant glass, ⅛ in. thick	10 " " " "
Maze glass (without wire), ⅛ in. thick	14 " " " "
Maze glass, 3/16 in. thick	16 " " " "
Prismatic glass in sheets	from 25 to 50 " " " "

PRICE-LIST OF POLISHED PLATE GLASS.

IN EFFECT SINCE 1894.

Sizes in inches; prices in dollars and cents.

Length	24	28	32	36	40	44	48	52	56	60
32	12.80	14.90	17.10	19.20	21.30	23.50	34.70	37.60	40.50	43 40
34	13.60	15.90	18.10	20.40	22.70	33.70	36.90	39.90	43.00	46.00
36	14.40	16.80	19.20	21.60	24.00	35.80	39.00	42.20	45.50	48.80
38	15.20	17.70	20.30	22.80	34.30	37.80	41.20	44.60	48.00	51.50
40	16.00	18.70	21.30	24.00	36.10	39.70	43.40	47.00	50.60	54.20
42	16.80	19.60	22.40	34.10	37.90	41.70	45.50	49.30	53.10	56.90
44	17.60	20.50	23.50	35.80	39.70	43.80	47.70	51.70	55.60	59.60
46	18.40	21.50	33.20	37.40	41.50	45.70	49.90	54.00	58.10	62.30
48	19.20	22.40	34.70	39.00	43.40	47.70	52.00	56.40	60.70	65.00
50	20.00	23.30	36.10	40.60	45.20	49.70	54.20	58.70	63.20	70.90
52	20.80	32.90	37.60	42.20	47.00	51.70	56.40	61.00	68.70	73.70
54	21.60	34.10	39.00	43.90	48.80	53.60	58.50	63.40	71.40	76.50
56	22.40	35.40	40.50	45.50	50.60	55.60	60.70	68.70	74.10	79.30
58	23.20	36.70	41.90	47.10	52.40	57.60	62.90	71.20	76.70	82.20
60	24.00	37.90	43.40	48.80	54.20	59.60	65.00	73.70	79.30	85.00
62	33.60	39.20	44.80	50.40	56.00	61.60	70.30	76.10	82.00	88
64	34.70	40.50	46.20	52.00	57.80	63.60	72.50	78.60	84.70	91
66	35.80	41.70	47.70	53.60	59.60	68.60	74.80	81.10	87.30	94
68	36.90	43.00	49.20	55.30	61.40	70.70	77.10	83.50	89.90	96
70	38.00	44.30	50.60	56.90	63.20	72.70	79.30	85.90	92.50	99
72	39.00	45.50	52.00	58.50	65.00	74.80	81.60	88.40	95.20	102
74	40.10	46.80	53.40	60.10	69.90	76.90	83.90	90.90	97.90	105
76	41.20	48.10	54.90	61.80	71.80	79.00	86.10	93.30	101	108
78	42.30	49.30	56.40	63.40	73.70	81.10	88.40	95.70	103	110
80	43.40	50.60	57.80	65.00	75.60	83.10	90.70	93.30	106	113
82	44.50	51.90	59.20	69.70	77.50	85.20	92.90	101	108	116
84	45.50	53.10	60.70	71.40	79.40	87.30	95.20	103	111	119
86	46.60	54.40	62.10	73.10	81.30	89.40	97.50	106	113	122
88	47.70	55.70	63.60	74.80	83.10	91.50	99.70	108	116	125
90	48.80	56.90	65.00	76.50	85.00	93.50	102	110	119	127
92	49.90	58.20	69.50	78.20	86.90	95.60	104	113	122	130
94	51.00	59.50	71.00	79.90	88.80	97.70	107	115	124	133
96	52.00	60.70	72.50	81.60	90.70	99.70	109	118	127	136
98	53.10	62.00	74.10	83.30	92.60	102	111	120	130	139
100	54.20	63.30	75.60	85.00	94.50	104	113	123	132	142
102	55.30	64.50	77.10	86.70	96.40	106	116	125	135	144
104	56.40	68.70	78.60	88.40	98.30	108	118	128	138	147
106	57.50	70.10	80.10	90.10	100	110	120	130	140	150
108	58.50	71.40	81.60	91.80	102	112	122	133	143	153
110	59.60	72.70	83.10	93.50	104	114	125	135	145	165
112	60.70	74.10	84.60	95.20	106	116	127	137	148	168
114	61.80	75.40	86.10	96.90	108	118	129	140	151	171
116	62.90	76.70	87.60	98.60	110	121	131	142	162	174
118	64.00	78.00	89.10	100	112	123	134	145	165	177
120	65.00	79.30	90.70	102	114	125	136	147	168	180
122	69.10	80.70	92.20	104	115	127	138	150	171	183
124	70.30	82.00	93.70	105	117	129	141	152	174	186
126	71.40	83.30	95.20	107	119	131	143	164	176	189
128	72.50	84.70	96.70	109	121	133	145	166	179	192
130	73.70	86.00	98.20	110	123	135	147	169	182	195
132	74.80	87.30	99.70	112	125	137	150	172	185	198
134	76.00	88.60	101	114	126	139	152	174	188	201
136	77.10	89.90	103	116	128	141	163	177	190	204
138	78.20	91.30	104	118	130	143	166	179	193	207

PRICE-LIST OF POLISHED PLATE GLASS—(*Continued*).

Sizes in inches; prices in dollars.

Length	62	64	66	68	70	72	74	76	78	80	82	84
62	91											
64	94	97										
66	97	100	103									
68	100	103	106	109								
70	102	106	109	112	116							
72	105	109	112	116	119	122						
74	108	112	115	119	122	126	129					
76	111	115	118	122	126	129	133	136				
78	114	118	121	125	129	133	136	140	144			
80	117	121	125	128	132	136	139	144	147	151		
82	120	124	128	132	136	139	143	147	151	164	168	
84	123	127	131	135	139	143	147	151	164	168	172	176
86	126	130	134	138	142	146	150	163	168	172	176	181
88	129	133	137	141	145	150	163	167	172	176	180	185
90	132	136	140	144	149	153	166	171	175	180	184	189
92	135	139	143	148	152	166	170	175	179	184	189	193
94	138	142	146	151	165	169	174	179	183	188	193	197
96	141	145	150	163	168	173	178	182	187	192	197	202
98	143	148	153	167	171	176	181	186	191	196	201	206
100	146	151	165	170	175	180	185	190	195	200	205	210
102	149	163	168	173	178	184	189	194	199	204	209	214
104	152	166	172	177	182	187	192	198	203	208	213	218
106	164	170	175	180	185	191	196	201	207	212	217	223
108	167	173	178	184	189	194	200	205	210	216	221	227
110	170	176	181	187	192	198	203	209	214	220	225	231
112	174	179	185	190	196	202	207	213	218	224	230	235
114	177	182	188	194	199	206	211	217	222	228	234	239
116	180	185	191	197	203	209	215	220	226	232	238	244
118	183	189	195	201	206	212	218	224	230	236	242	248
120	186	192	198	204	210	216	222	228	234	240	246	252
122	189	195	201	207	213	220	226	232	238	244	250	256
124	192	198	205	211	217	223	230	236	242	248	254	260
126	195	202	208	214	220	227	233	239	246	252	258	265
128	198	205	211	218	224	230	237	243	250	256	262	269
130	201	208	214	221	227	234	240	247	253	260	266	273
132	205	211	218	224	231	238	244	251	257	264	271	277
134	208	214	221	228	234	241	248	255	261	268	275	281
136	211	218	224	231	238	245	251	258	265	272	279	286
138	214	221	228	234	241	248	255	262	269	276	283	290
140	217	224	231	238	245	252	259	266	273	280	287	294
142	220	227	234	241	248	256	263	270	277	284	291	298
144	223	230	238	245	252	259	266	274	281	288	295	302
146	226	234	241	248	255	263	270	277	285	292	299	307
148	229	237	244	252	259	266	274	281	289	296	303	311
150	232	240	247	255	262	270	277	285	292	300	307	315
152	236	243	251	258	266	274	281	289	296	304	312	319
154	239	246	254	262	269	277	285	293	300	308	316	323
156	242	250	257	265	273	281	288	296	304	312	320	328
158	245	253	261	269	276	284	292	300	308	316	324	332
160	248	256	264	272	280	288	296	304	312	320	328	336
162	251	259	267	275	283	292	300	308	316	324	332	340
164	254	262	271	279	287	295	303	312	320	328	336	344
166	257	266	274	282	290	299	307	315	324	332	340	349
168	260	269	277	286	294	302	311	319	328	336	344	353

PRICE-LIST OF POLISHED PLATE GLASS—(*Continued*).

Sizes in inches; prices in dollars.

Length	86	88	90	92	94	96	98	100	102	104	106	108
90	193	198	202									
92	198	202	207	212								
94	202	207	211	216	221							
96	206	211	216	221	226	230						
98	211	216	220	225	230	235	240					
100	215	220	225	230	235	240	245	250				
102	219	224	229	235	240	245	250	255	260			
104	224	229	234	239	244	250	255	260	265	270		
106	228	233	238	244	249	254	260	265	270	276	281	
108	232	237	243	248	254	259	265	270	275	281	286	292
110	237	242	247	253	258	264	269	275	280	286	291	297
112	241	246	252	258	263	269	274	280	285	291	297	302
114	245	251	256	262	268	274	279	285	290	296	302	308
116	250	255	261	267	273	278	284	290	296	302	307	313
118	254	260	265	272	277	283	289	295	301	307	313	319
120	258	264	270	276	282	288	294	300	306	312	318	324
122	262	268	274	281	287	293	299	305	311	317	323	329
124	267	273	279	285	291	298	304	310	316	322	329	335
126	271	277	283	290	296	302	309	315	321	328	334	340
128	275	282	288	294	301	307	314	320	326	333	339	346
130	279	286	292	299	305	312	318	325	331	338	345	351
132	284	290	297	304	310	317	323	330	337	343	350	356
134	288	295	301	308	315	322	328	335	342	348	355	402
136	292	299	306	313	320	326	333	340	347	354	400	408
138	297	304	310	317	324	331	338	345	352	359	406	414
140	301	308	315	322	329	336	343	350	357	404	412	420
142	305	312	319	327	334	341	348	355	402	410	418	426
144	310	317	324	331	339	346	353	360	408	416	424	432
146	314	321	328	336	343	350	358	406	414	422	430	438
148	318	326	333	340	348	355	403	411	419	428	436	444
150	322	330	337	345	352	360	408	417	425	433	442	450
152	327	334	342	350	357	405	414	422	431	439	448	456
154	331	339	346	354	402	411	419	428	436	445	453	462
156	335	343	351	359	407	416	425	433	442	451	459	468
158	340	348	355	404	413	421	430	439	448	456	465	474
160	344	352	360	409	418	427	436	444	453	462	470	480
162	348	356	405	414	423	432	441	450	459	468	477	759
164	353	401	410	419	428	437	446	456	465	474	755	769
166	357	406	415	424	433	443	452	461	470	480	764	778
168	401	411	420	429	439	448	457	467	476	758	773	787
170	406	416	425	434	444	453	463	472	753	767	782	797
172	411	420	430	440	449	459	468	478	761	776	791	806
174	416	425	435	445	454	464	474	755	770	785	801	979
176	420	430	440	450	460	469	479	764	779	794	810	990
178	425	435	445	455	465	475	757	773	788	803	983	1001
180	430	440	450	460	470	480	766	781	797	812	994	1012
182	478	489	500	511	523	758	774	790	806	986	1005	1024
184	483	495	506	517	751	767	783	799	977	997	1016	1035
186	489	500	511	523	759	775	791	807	988	1007	1027	1046
188	494	506	517	751	767	783	800	979	999	1018	1038	1057
190	500	511	522	759	775	792	808	990	1009	1029	1049	1069
192	688	704	720	920	940	960	980	1000	1020	1040	1060	1080
194	695	711	909	930	950	970	990	1010	1031	1051	1071	1091
196	703	719	919	939	960	980	1000	1021	1041	1062	1082	1102

PRICE-LIST OF POLISHED PLATE GLASS—(*Continued*).

Size in inches; prices in dollars.

Length	110	112	114	116	118	120	124	128	132	136	140	144
110	302	308	314	319	324	330	341	352	403	415	428	440
112	308	348	355	361	367	373	386	398	411	423	436	448
114	314	355	361	367	374	380	393	405	418	431	443	456
116	319	361	367	374	380	387	400	412	425	438	451	464
118	324	367	374	380	387	393	406	420	433	446	459	472
120	330	373	380	387	393	400	413	427	440	453	467	480
122	335	380	386	393	400	407	462	478	493	507	522	762
124	341	386	393	400	406	413	470	485	501	516	753	775
126	346	392	399	406	413	420	477	493	508	524	766	787
128	352	398	405	412	420	427	485	501	516	756	778	800
130	357	404	412	419	426	433	493	509	525	767	790	812
132	403	411	418	425	433	440	501	516	847	873	898	990
134	409	417	424	432	439	447	509	524	860	886	977	1005
136	415	423	431	438	446	453	516	756	873	899	992	1020
138	422	429	437	445	452	460	523	767	886	977	1006	1035
140	428	436	443	451	459	467	753	778	898	992	1021	1050
142	434	442	450	457	465	473	764	789	976	1006	1035	1420
144	440	448	456	464	472	480	775	800	990	1020	1050	1440
146	446	454	462	470	479	760	786	811	1004	1034	1065	1460
148	452	460	469	477	758	771	796	987	1017	1048	1079	1480
150	458	467	475	755	768	781	807	1000	1031	1062	1094	1500
152	464	473	752	765	778	792	982	1013	1045	1077	1108	1520
154	471	479	762	775	789	802	995	1027	1059	1091	1123	1540
156	477	758	772	785	799	812	1007	1040	1072	1105	1327	1560
158	754	768	782	795	809	987	1020	1053	1086	1119	1344	1580
160	764	778	792	805	983	1000	1033	1067	1100	1322	1361	1600
162	773	787	802	975	996	1012	1046	1080	1114	1339	1378	1620
164	783	797	812	991	1008	1025	1059	1093	1315	1355	1395	1640
166	793	807	986	1003	1020	1037	1072	1107	1331	1372	1412	1660
168	802	980	997	1015	1032	1050	1085	1120	1347	1388	1429	1680
170	812	992	1009	1027	1045	1062	1098	1322	1364	1405	1446	1700
172	985	1003	1021	1039	1057	1075	1111	1338	1380	1421	1463	1720
174	997	1015	1033	1051	1069	1087	1124	1353	1396	1438	1480	1740
176	1008	1027	1045	1063	1082	1100	1326	1369	1412	1454	1497	2200
178	1020	1038	1057	1075	1094	1112	1341	1384	1428	1471	1514	2225
180	1031	1050	1069	1087	1106	1125	1356	1400	1444	1487	1531	2250
182	1043	1062	1081	1100	1119	1327	1371	1416	1460	1504	2212	2275
184	1054	1073	1092	1112	1319	1342	1386	1431	1476	1521	2236	2300
186	1066	1085	1104	1124	1334	1356	1401	1447	1492	2196	2260	2325
188	1077	1097	1116	1325	1348	1371	1417	1462	1508	2219	2285	2350
190	1089	1108	1316	1339	1362	1385	1432	1478	1524	2243	2309	2375
192	1100	1120	1330	1353	1376	1400	1447	1493	2200	2267	2333	2400
194	1111	1320	1344	1367	1391	1415	1462	1509	2223	2290	2358	2425
196	1123	1334	1358	1382	1405	1429	1477	1524	2246	2314	2382	2450
198	1323	1347	1372	1396	1419	1444	1492	2200	2269	2337	2406	2475
200	1337	1361	1385	1410	1434	1458	1507	2222	2292	2361	2431	2500
202	1350	1375	1399	1424	1448	1473	1522	2244	2315	2385	2455	
204	1364	1388	1413	1438	1463	1487	2196	2267	2337	2408	2479	
206	1377	1402	1427	1452	1477	1502	2217	2289	2360	2432		
208	1390	1416	1441	1466	1491	1517	2239	2311	2383	2456		
210	1404	1429	1455	1480	1506	1531	2260	2333	2406	2479		
212	1417	1443	1469	1494	1520	2208	2282	2356	2429			
214	1430	1456	1482	1508	2192	2229	2303	2378	2452			
216	1444	1470	1496	1522	2212	2250	2325	2400	2475			

Faboleum.* —During the past few years this material has been introduced as a substitute for glass, particularly for skylights of large area, and in all places where the breakage of glass constitutes an element of trouble, expense, or danger. It consists of wire cloth, embedded in a translucent impervious material, which is strong and durable, flexible and elastic, weather-proof and unbreakable. It is a non-conductor, is easily cleaned, and is a better protection against fire than glass. It transmits a large amount of light and diffuses it well.

The fabric is not damaged in the slightest by rain or snow, cold or heat. The leaks that develop in glass skylights, due to the expansion and contraction of the framework under the action of the weather, do not trouble faboleum because it is flexible and yielding.

Faboleum is of a pale amber color, and the light it transmits is of somewhat the same tint, being very soft and pleasant to work under. The amount of light transmitted is not quite equal to that transmitted by ordinary skylight glass, but, on the other hand, it is better diffused. "Where one quarter of the roof is covered with fabric the lighting is practically perfect."† The fabric is manufactured in sheets 3′ 3″ wide and in lengths from 4′ 6″ to 9′ 0″. The cost is from 13 to 15 cts. per square foot at the factory at Quincy, Mass. The framework for faboleum is best made of wood, to which the fabric is nailed. Wooden skylights covered with this fabric cost complete from 25 to 30 cts per square foot.

Faboleum was employed in the buildings of the Tennessee Centennial Exposition held at Nashville in 1897, and nearly 100,000 sq. ft. were used in the principal buildings of the Trans-Mississippi and International Exposition held in Omaha, Neb., in 1898.

Mirrors.

Mirrors are made by silvering the back of glass. Polished plate glass is the only kind that is suitable for mirrors. The price of mirrors is based on the price of the glass plus the cost of silvering.

Kinds of Mirrors.—"There are two kinds of mirrors on the market, one the old time reliable mercury-back mirror, the other the nitrate of silver, or what is better known to the trade as the patent-back mirror. The latter is now and has, in recent years, been most extensively sold as a substitute for the former.

In the manufacture of mercury-back mirrors no chemicals are used, only two metals, mercury and tin-foil. The affinity of mercury for tin forms an amalgam impervious to and not affected by the atmosphere.

A mercury-back mirror is universally considered to be the only durable and permanent mirror.

A nitrate-of-silver or patent-back mirror is produced by the precipitation of a chemical solution of nitrate of silver and

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* Manufactured by the Translucent Fabric Company, Quincy, Mass.
† M. S. Ketchum.

other media on the surface of the glass, to which is added one coat of shellac varnish overlaid with one or more coats of paint. This mirror, irrespective of the quality of the glass from which it is made, will steadily deteriorate from the date of its manufacture to that of its final collapse, which may occur at any time from a few months, but certainly within a few years."

MEMORANDA ON ROOFING.

Shingles.*—The best shingles are those made from cypress, redwood, or cedar, in the order mentioned. Redwood, while perhaps not quite as durable as cypress, is less inflammable; sawed pine shingles are inferior to cedar, and spruce shingles are not suitable for good work.

Cypress shingles are usually 18 ins. long and 7/16 in. thick at the butt. Those from all other woods are 16 ins. long, and about 5/16 in. thick at the butt.

Ordinary roofing shingles are of random widths, varying from 2½ to 14 and sometimes 16 ins. They are put up in bundles, usually four to the thousand. A "thousand" common shingles means the equivalent of 1,000 shingles 4 ins. wide.

Dimension Shingles are sawn to uniform width, either 4, 5, or 6 ins. Dimension shingles with the butt sawn to various patterns are also carried in stock.

NUMBER OF SQUARE FEET 1000 SHINGLES WILL COVER.†

Laid.	Area Covered.	No. to a Square.
4″ to the weather	100 sq. ft.	1,000
4¼″ " " "	110 " "	910
4½″ " " "	120 " "	833
5″ " " "	133 " "	752
5½″ " " "	145 " "	690
6″ " " "	157 " "	637

On hip roofs, or for four valleys, add 5 per cent. for cutting. On irregular roofs with dormer windows, add 10 per cent. It is claimed that redwood shingles will go farther than cedar shingles.

With a rise to the roof of 8 to 10 ins. to the foot, cedar shingles should be laid 4 to 4¼ ins. to the weather; with rise from 10 to 12 ins., 4¼ to 4⅜ ins. to the weather; and on steeper roofs they may be laid 4½ to 5 ins. Redwood shingles may be laid ½ in. more to the weather.

On walls cedar shingles are commonly laid 5 ins. to the weather, and redwood shingles 6 ins.

Labor.—An average shingler should lay 1,500 shingles in 9 hours on plain work; on irregular roofs with dormers, 1,000 per 9 hours.

It requires about 5 lbs. of threepenny or 7½ lbs. of fourpenny nails to 1,000 shingles.

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* For more complete information see Part II, Building Construction and Superintendence, pp. 190–199.

† These figures are intended to allow for some waste.

Cost.—Common cedar shingles of the best grade cost from \$2.25 to \$3.50 per M, according to locality. Redwood shingles cost from \$4 to \$5. In Denver, shingles are laid under contract for from \$1.25 to \$2.00 per square, all materials furnished.

Slate Roofs.

Characteristics of Good Slate.—A good slate should be both hard and tough.

If the slate is too soft, however, the nail-holes will become enlarged and the slate will become loose. If it is too brittle the slate will fly to pieces in the process of squaring and holing and will be easily broken on the roof. "A good slate should give out a sharp metallic ring when struck with the knuckles; should not splinter under the slater's axe; should be easily 'holed' without danger of fracture, and should not be tender or friable at the edges."

The surface when freshly split should have a bright metallic lustre and be free from all loose flakes or dull surfaces.

Most slates contain ribbons or seams which traverse the slate in approximately parallel directions. Slates containing soft ribbons are inferior and should not be used in good work.

Color.—The color of slates varies from dark blue, bluish black, and purple to gray and green. There are also a few quarries of red slate.* The color of the slate does not appear to indicate the quality. The red and dark colors are generally considered the most effective, and the greens are generally used only on factories, storehouses, and buildings where the appearance is not of so much importance.

Some slates are marked with bands or patches of a different color, and the dark-purple slates often have large spots of light green upon them. These spots do not as a rule affect the durability of the slate, but they greatly detract from its appearance.

Grading of Slates.—The *Brownville, Maine,* slates are graded as follows: No. 1. Every sheet to be full $\frac{3}{16}''$ thick, both sides smooth and all corners full and square. No pieces to be winding or warped.

No. 2. Thickness may vary from $\frac{1}{8}''$ to $\frac{1}{4}''$, all corners square, one side generally smooth, one side generally rough, no badly warped slates.

The *Bangor, Penn.,* slates are graded:

No. 1 *Clear.*—A pure slate without any faults or blemishes.

No. 1 *Ribbon.*—As well made as No. 1 Clear, except that it contains one or more "ribbons" (a black band or streak across the slate), which, however, are high enough on the slate to be covered when laid, thus presenting a No. 1 roof.

No. 2 *Ribbon.*—This contains several "ribbons," some of which cannot be covered when laid.

No. 2 *Clear.*—A slate without "ribbons," made from rough beds.

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* The best red slates are believed to be those quarried by the Algonquin Red Slate Company of Worcester, Mass., and Mathews' unfading bright red, the Alden Speare's Sons Company of New York City selling agents.

Hard Beds.—A clear Bangor slate, not quite as smooth as No. 1 Clear, but much better than a No. 2 Clear.

Ordinary Bent Slate.—A smooth slate similar to No. 1 Clear, but bent at a radius of about 12 ft.

Punching.—Formerly slates were punched for nail-holes on the job; now, however, slates are bored and countersunk at the quarry, when so ordered. Architects should always specify that "slates be bored and countersunk," as punching badly damages the slates.

Sizes.—The sizes of slates range from $9'' \times 7''$ to $24'' \times 14''$, there being some thirty-seven different sizes; the more common sizes, however, are those given in the following table.

The sizes of slates best adapted for plain roofs are the large wide slate, such as $12'' \times 16''$, $18'' \times 12''$, $20'' \times 12''$, or $24'' \times 14''$; the large sizes make less joints in the roof, require less nails, and avoid small pieces at hips and valleys. For roofs cut up into small sections the smaller sizes, such as $14'' \times 7''$ or $16'' \times 8''$, look the best.

Thickness.—Slates vary in thickness from $\frac{1}{8}$ to $\frac{3}{8}$ in.; $\frac{3}{16}$ in. is the usual thickness for ordinary sizes (see Grading of Slates).

Laying.—Slates are laid either on a board sheathing (rough or tongued and grooved) covered with tarred or water-proof paper or felt, or on roofing-laths 2 to 3 ins. wide and from 1 to $1\frac{1}{4}$ ins. thick, nailed to the rafters at distances apart to suit the gauge of the slates. Each slate should lap the slate in the second course below 3 ins.

The slates are fastened with two threepenny or fourpenny nails, one near each upper corner. For slates $20'' \times 10''$ or larger, fourpenny nails should be used. Copper, composition, tinned, or galvanized nails should be used. Plain iron nails are speedily weakened by rust, break, and allow the slates to be blown off.

On iron roofs slates are often placed directly on small iron purlins spaced at suitable distance to receive them, and fastened with wire or special forms of fasteners.

The Gauge of a slate is the portion exposed to the weather, which should be one half of the remainder obtained by subtracting 3 ins. from the length of the slate.

Roofs to be covered with slate should have a rise of not less than 6 ins. to the foot for 20- or 24-in. slates, or 8 ins. for smaller sizes.

Elastic Cement.—In first-class work, the top course of slate on ridge, and the slate for 2 to 4 ft. from all gutters and 1 ft. each way from all valleys and hips, should be bedded in elastic cement.

Flashings.—By "flashings" are meant pieces of tin, zinc, or copper laid over slate and up against walls, chimneys, copings, etc.

Counter-flashings are of lead or zinc, and are laid between the courses in brick, and turned down over the flashings. In flashing against stonework, grooves or reglets often have to be cut to receive the counter-flashings.

Close and Open Valleys.—A close valley is where the slates

are mitred and flashed in each course and laid in cement. In such valleys no metal can be seen. Close valleys should only be used for pitches above 45°.

An open valley is where the valley is formed of sheets of copper or zinc 15 or 16 ins. wide, and the slates laid over these.

Measurement.—Slates are sold by the "square," by which is meant a sufficient number of slates of any size to cover 100 sq. ft. of surface on a roof, with 3 ins. of lap, over the head of those in the second course below. The square is also the basis on which the cost of laying is measured.

"Eaves, hips, valleys, and cuttings against walls or dormers are measured extra—1 ft. wide by their whole length, the extra charge being made for waste material and the increased labor required in cutting and fitting. Openings less than 3 sq. ft. are not deducted, and all cuttings around them are measured extra. Extra charges are also made for borders, figures, and any change of color of the work and for steeples, towers, and perpendicular surfaces." *

Cost.—The cost of slates varies with the size, color, and quality. The prices given in the following table are about the average for blue-black slate, of No. 1 grade, at the quarry. It will be seen that the *medium* sizes cost the most, and the larger and smaller sizes the least. The larger sizes make the cheapest roof.

Red slates cost from 60 to 150 per cent. more than black slates.

NUMBER AND COST OF SLATES, AND POUNDS OF NAILS TO 100 SQUARE FEET OF ROOF.

(3-inch Lap.)

Sizes of Slate.	Exposed when Laid.	Number to a Square.	Weight of Galvanized Nails.			Cost per Square at Quarry.
ins.	ins.			lbs.	oz.	
14×24	10½	98	4d.	1	6	$6.10
12×24	10½	115	4d.	1	10	6.60
12×22	9½	126	4d.	1	12	6.50
11×22	9½	138	4d.	1	15	6.90
12×20	8½	142	4d.	2	0	6.80
10×20	8½	170	4d.	2	6	6.80
12×18	7½	160	3d.	1	13	6.80
10×18	7½	192	3d.	2	3	7.20
9×18	7½	214	3d.	2	7	7.10
12×16	6½	185	3d.	2	2	6.80
10×16	6½	222	3d.	2	8	7.10
9×16	6½	247	3d.	3	0	7.00
8×16	6½	277	3d.	3	2	7.20
10×14	5½	262	3d.	3	0	6.60
8×14	5½	328	3d.	3	12	6.60
7×14	5½	374	3d.	4	4	6.40
8×12	4½	400	3d.	4	9	5.50
7×12	4½	458	3d.	5	3	5.00
6×12	4½	533	3d.	6	1	4.80

* The Building Trades Pocket-book

The cost of blue-black slate roofs, complete, varies from $7 to $13 per square, depending on the class of work and remoteness from the quarries.

The additional cost of laying slate in elastic cement varies from $1.50 to $2 per square.

An experienced roofer will lay on an average two squares of slate in ten hours.

Weight.—Slate roofing $\frac{3}{16}$ in. thick will weigh on the roof about $6\frac{1}{2}$ lbs. per square foot, and $\frac{1}{4}$ in. slates $8\frac{3}{4}$ lbs., the smaller sizes weighing the most on account of the lap. The actual weight of a square foot of slate $\frac{1}{4}$ in. thick is 3.63 lbs.

Roofing Tile.

The term roofing tile is commonly understood to refer to exterior roof covering made from clay with overlapping edges. Clay or terra-cotta roof tiles have long been very largely used in Europe, where their cost is much less than in America.

Since the year 1893 the advance here in the character and extent of roofing tile has been marked and rapid. This material can now be had for half the prices prevailing twelve years ago, and the result has been that thousands of squares of terra-cotta tiles have been placed on shops and factories which would under former conditions have been covered with slate or metal.

Whether or not a tile roof is as durable and satisfactory as one of No. 1 slate is a much-disputed question. The author is of the opinion that, considering the quantities used, slates have given better satisfaction than tile.

A tile roof, however, is certainly more attractive than a slate roof, and the author believes that there are many roofing tiles on the market which if properly laid will prove as tight and durable as slate.

There are so many patterns of roofing tile that it is impossible here to enter into a description of them. Of the various patterns, those which interlock are considered to make the most satisfactory roof from a practical standpoint.

Some manufacturers of roofing tile, notably the Ludowici Roofing Tile Company, make *glass tiles*, of the same pattern as the clay tiles, so that they may be worked in with them and used in place of skylights. Many thousands of these glass tiles have been used on the roofs of train-sheds, shops, and factories.

Roofing tile may be laid on felt or sheathing, or those with a proper interlocking device may be laid direct on wood or steel purlins without sheathing or inner roof of any kind. When so laid, to prevent the entrance of dust or dry snow, the joints should be pointed on the under side *after laying*. Most tiles, particularly of the older patterns, are nailed to the sheathing, but this is a defective principle of fastening and is superseded by the modern practice of fastening with copper wires from a pierced lug toward the *lower end* of the tile.

Roofing tiles weigh from 750 to 1,200 lbs. per square (100 sq. ft.).

The prices of tiles vary from \$6 to \$30 per square, according to pattern and finish.

The cost of laying varies from \$1.50 to \$5 per square, according to the pattern of tile used and the character and extent of the roof.

The principal manufacturers of roofing tile in this country are the Akron Roofing Tile Company, Akron, Ohio; Celadon Roofing Tile Company, New York, N. Y.; C. A. Conway & Co., New Philadelphia, Ohio; Federal Roofing and Tile Company, St. Louis, Mo.; Ludowici Roofing Tile Company, Chicago, Ill.; Mound City Roofing Tile Company, St. Louis, Mo.; National Tile Roofing Company, Lima, Ohio; Ohio Roofing Tile Company, Ottawa, Ohio—from whom catalogues giving full information may be obtained.

Sheet-metal Tiles.—Roofing tiles stamped from sheet steel, plain or galvanized, and also from sheet copper, in imitation of clay tiles, are made by several parties, notably Merchant & Company of Philadelphia and W. H. Mullins of Salem, Ohio, and have been extensively used.

The first cost of these tiles (except those of copper) is much less than that of clay tiles and they do not require as heavy roof framing. Tin or galvanized-iron tiles, however, must be painted every few years, so that for a long period of years they will probably cost as much as clay tiles and more than slate.

Galvanized-iron tiles of the "Spanish" pattern cost from \$13 to \$15 per square laid and painted, and ordinary tin shingles from \$8 to \$10.

Tin Roofs.

The Sheets.—Roofing plates are made of soft steel or wrought iron (more commonly of the former) and covered with a mixture of lead and tin, and are designated as "terne plates," in distinction from plates coated only with tin and therefore called "bright tin." Roofing plates are coated by two methods. The original manner of coating the plates (commonly designated "Old Process") was by dipping the black plates by hand into the mixture of tin and lead, and allowing the sheets to absorb all the coating that was possible; and several brands of roofing tin are still made by this process. The other process, by which the majority of roofing plates are now made, is known as the "Patent Roller Process," by which the plates are put into a bath of tin and lead, and are passed through rolls, the pressure of which leaves on the iron or steel a thickness of coating which, to a great extent, determines the value of the plate. These rolls can be so adjusted as to leave a good amount of coating on the plate, an ordinary coating, or a very scant one; the heavier the coating the more valuable the plate.

It is claimed that hand-dipped plates will last much longer than those made by the new process, although the latter process

is much more extensively used and many good roofing sheets are made by it.

The best roofing plates always have the brand stamped on them, and as the manufacturers have a pecuniary interest in keeping up the reputation of these brands, the only way of being sure of a good tin roof is to specify a brand of tin that has a reputation for quality and durability. Some of the best known brands are Taylor's "Old Style," Merchant's "Old Method," "M F," "Scott's Extra Coated," "Margaret," and "Admiral."

Sizes.—The common sizes of tin plates are 10×14 ins. and multiples of that measure. The sizes more generally used are 14×20 ins. and 20×28 ins. The larger size is the more economical to lay, and hence roofers prefer to use it, but for flat roofs the 14×20 size makes the better roof.

Thicknesses.—Terne plates are made in two thicknesses, viz., I C, in which the iron body weighs about 50 lbs. per 100 sq. ft., and I X, in which it weighs $62\frac{1}{2}$ lbs. per 100 sq. ft. For roofing the I C, or lighter weight, is to be preferred, because the seams will not suffer as much from contraction and expansion as with the thicker plates.

For spouts, valleys, and gutters, however, I X plates should always be specified, and should preferably be used for flashings, as they are *stiffer* and less liable to be dented or punched. The thickness of the iron does not add to the *durability* of the plates, as this depends entirely upon the tin coating.

Weights.—The standard weight of 14×20 in. I C terne plates is 107 lbs. to 112 sheets (the number usually packed in one box), and of 14×20 in. I X sheets, 135 lbs. 20×28 in. sheets should weigh just twice as much. The black sheets before coating should weigh, per 112 sheets, from 95 to 100 lbs. for I C, 14×20 ins., and from 125 to 130 lbs. for I X, 14×20 ins. The difference between the weight of the black sheets and of the finished sheets shows the weight of the tin. A heavily coated tin should weigh from 115 to 120 lbs. per 112 sheets for I C, 14×20 ins., and from 145 to 150 lbs. for I X, 14×20 ins. 20×28 in. sheets should, of course, weigh twice as much.

The Roof.—Roofs with less than one third pitch are made with flat seams and should preferably be covered with sheets 14×20 ins., rather than from sheets 20×28 ins., because the larger number of seams stiffen the surface and help to prevent buckles and rattling in stormy weather. For a flat-seam roof 1-in. barbed and tinned roofing nails should be used, not over 6 ins. apart, well under the edge. They should be well covered up and the seams should be pounded down over the edge with a wooden mallet. Nails must never be exposed. The seams should be made with great care; sufficient time must be taken to properly "sweat" the solder into the seams.

Steep tin roofs should be made with *standing seams* and from sheets 20×28 ins. The sheets are first double-seamed and soldered together, preferably end to end, into long strips that reach from eaves to ridge. The sloping seams are composed of

two "upstands," interlocked and held in place by cleats. The standing seams are not soldered, but are simply locked together with the cleats folded in from 15 to 18 ins. apart. Nails should be driven into the cleats only.

The *use of acid* in soldering seams in a tin roof is to be carefully avoided; acid coming in contact with the bare iron on the cut edges and corners where the sheets are folded and seamed together will cause rusting. No other soldering flux but good rosin should ever be used.

Durability.—A tin roof of good material, properly put on, and kept properly painted, will last from thirty to forty years. It should not be painted for the first time until it has been well washed by rain, to get the grease off the tin; and all lumps or rosin left on the roof should be removed as soon as the tin is laid and soldered. One or more layers of felt-paper should be placed under the tin, to serve as a cushion, and also to deaden the noise produced by rain striking the tin.

The durability of tin roofing, and especially of gutters, valleys, and flashings, is generally increased by *painting the tin* on the back before laying. An excellent paint for tin roofs is composed of 10 lbs. Venetian red, 1 lb. red lead, 1 gallon pure linseed-oil.

Number of Sheets Required to a Square.—*For flat-seam roofing* a sheet of tin 14×20 ins. with $\frac{1}{2}$-in. edges, measures when edged or folded 13×19 ins., or 247 sq. ins.; consequently the number of sheets required to a square equals $14{,}400 \div 247$, or $58\frac{1}{2}$. 1,000 sq. ft. requires 583 sheets. A box of 112 sheets 14×20 ins. will cover approximately 192 sq. ft.

Sheets 20×28 ins. measure when edged or folded 19×27 ins., or 513 sq. ins. To cover 1,000 sq. ft. (10 squares) requires 288 sheets.

The standing seams and locks on a steep roof require $2\frac{3}{4}$ ins. off the width and $\frac{3}{4}$ in. off the length of the sheet. A sheet 20×28 ins. with the seams on the narrow edges will cover 486 sq. ins., and with the seams on the long edges 470 sq. ins. The former requires 297 sheets to 1,000 sq. ft , and the latter 307 sheets.

The cost of tin roofing varies from $8 to $11 per square, according to the grade of tin used and the scale of wages. Standing-seam roofs cost about 50 cts. a square less than flat-seam roofs.

Slag or Gravel Roofing (Composition Roofing).

The ordinary gravel roofing is formed by first covering the surface of the roof with dry felt (paper) and over this laying three, four, or five layers of tarred or asphaltic felt, the layers of felt lapping each other like shingles, so that only from 6 to 10 ins. of each layer are exposed.

Flashing against walls, chimneys, curbs of skylights, etc., is done by turning the felt up 4 ins. against the wall. Over this is laid an 8-in. strip with half its width on the roof. The upper edge of the strip and of the several layers of felt is then fastened to the wall by nailing wooden strips or laths over the felt and into the wall.

A better method is to lay two plys of tarred felt lapping each other 17 ins. and then spreading a coat of pitch over the entire roof. On this again three more layers of felt are laid, then coated with pitch, into which the crushed slag or screened gravel is embedded. Each layer of felt lapping another should be mopped 2 ins. more than its exposed surface back from the edge.

The following specification prepared by the Barrett Manufacturing Company describes the latter method, as also the materials that should be used to secure a first-class job:*

Specification for Slag or Gravel Roofing.†

Over the entire roof shall be laid a five- (5) ply coal-tar pitch felt and slag or gravel roof, to be constructed as follows:

The rosin-sized sheathing paper to be used shall weigh not less than six (6) lbs. per 100 sq. ft.

The felt shall weigh not less than fourteen (14) lbs. per 100 sq. ft., single thickness.

The pitch shall be the best quality of straight-run coal-tar pitch distilled direct from American coal-tar, and there shall be used not less than one hundred and twenty (120) lbs. (gross weight) per 100 sq. ft. of completed roof.

The nailing shall be done with threepenny barbed-wire roofing nails driven through tin discs.

The slag or gravel shall be of such a grade that no particles shall exceed five eighths ($\frac{5}{8}$) of an inch or be less than one fourth ($\frac{1}{4}$) of an inch in size. It shall be dry and free from dust or dirt. In cold weather it must be heated immediately before using. Not less than three hundred (300) lbs. of slag or four hundred (400) lbs. of gravel shall be used per 100 sq. ft.

The materials shall be used as follows:

First lay one thickness of rosin-sized sheathing paper (A), lapping each sheet 1 in. over the preceding one, and nailing only so often as may be necessary to hold in place until covered with the tarred felt, and the nailing may be omitted entirely if practicable.

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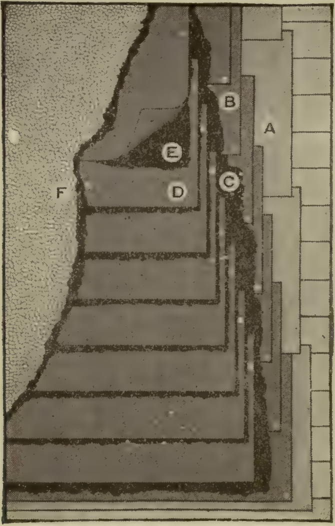
* For specifications for ordinary gravel roofing, including flashing, see Part II, Building Construction and Superintendence, p. 498.

† Known as Barrett's specifications.

Over the rosin-sized sheathing lay two (2) full thicknesses of tarred felt (B), lapping each sheet seventeen (17) ins.* over the preceding one, and nailing along the exposed edges of the sheets only so often as may be necessary to hold the sheets in place until the remaining felt can be applied.



Gravel Roofing, showing Method of Laying.

Over the entire surface of the felt thus laid spread a uniform coating of pitch (C), mopped on. Then lay three (3) full thicknesses of felt (D), lapping each sheet twenty-two (22) ins. over the preceding one, and nailing, as laid, every three (3) ft., not more than ten (10) ins. from the upper edge.

When the felt is thus laid and secured, mop with pitch (E) the full width of twenty (20) ins. under each lap. Then spread over the entire surface of the roof a uniform coating of pitch, into which, while hot, embed slag or gravel (F).

Note.—When this roof is to be laid over hydraulic cement concrete, as in fire-proof construction, omit the rosin sheathing paper, and in its place coat the concrete with hot pitch.

The only difference between slag and gravel roofing is that for the former crushed slag is used instead of gravel.

As there are several different weights of tarred felt, the specifications should either give the weight per 100 sq. ft. or the number of some particular brand, as Barrett's No. 1, 2, or 3.

Temporary roofs may be made with three or even two thicknesses of tarred felt.

The object of laying dry felt or rosin-sized paper on the sheathing is to prevent the pitch from dripping through the cracks. The minimum weight of tarred felt that should ever be used on temporary jobs is 12 lbs. per 100 sq. ft. and the

* The width of roofing felts is 32 ins. A lap of 17 ins. gives a 2-in. "head-cover."

minimum amount of pitch 70 lbs. for a 3-ply roof and 90 lbs. for a 5-ply (*i.e.*, four layers of tarred felt.)*

Gravel roofs should not have a pitch of less than $\frac{3}{8}$ nor more than $\frac{3}{4}$ in. to the foot in hot climates, or in the Mountain States. In cold and damp climates the pitch may be as great as 4 ins. to the foot, but is not as desirable as one of $\frac{5}{8}$ in. to 1 in.

Fire-resisting Qualities.—While it cannot be called fireproof, it has been proved by carefully conducted tests that gravel roofing will protect a wooden roof better than tin.

The effect of fire on gravel roofing is to soften the asphalt and pitch in the roofing, to burn out the inflammable oil in the same, and to cause the residue to swell and form a porous, incombustible coke.

Durability.—A 3-ply gravel roof of 12-lb. felt and 70 lbs. of straight-run distilled pitch should last for from four to seven years; an ordinary 6-ply 15-lb. felt and 100 lbs. pitch from nine to ten years, and a roof put on as specified above, fifteen to twenty years, and under favorable circumstances even longer.

Tar roofing is not readily attacked by corrosive gases and will consequently last longer than metal on buildings exposed to such gases. Creosote-oil is often added to coal-tar pitch, particularly in cold weather, to make it run well and to make the slag or gravel stick. It is generally considered to lessen the life of the pitch.

Roofers generally give a five-year guarantee with gravel roofs.

Cost.—The cost of coal-tar gravel roofing varies with the times and locality from $2.50 to $3.50 per square for 3-ply, $3 to $5 for ordinary 5-ply, and about $7 for a roof as above specified.

Asphalt Roofing differs from coal-tar roofing principally in the substitution of asphalt or asphaltic cement for the coal-tar pitch, for saturating the felt as well as for mopping and surface coating.

It is claimed that the oils of asphalt do not evaporate as quickly as do those of coal-tar pitch under ordinary temperatures and that therefore the flexibility and life of asphaltic felts and coatings are not as quickly destroyed. As a matter of fact,

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* In the Western States the number of "ply" is construed to mean the total number of layers, including dry as well as saturated felt, and the terms 3 ply, 5 ply, etc., are hereinafter used on that basis. In the Eastern States, 3 ply, 5 ply, etc., usually refers to the number of layers of saturated felt. The total number of layers should always be specified.

asphalt roofs do not always last longer than some coal-tar roofs, but the chances are that they will last fully as long and possibly longer, depending upon the quality of the materials and the workmanship.

The asphalt used for roofing is obtained principally from the island of Trinidad.

The asphalt-roofing materials manufactured by the Warren Chemical and Manufacturing Company of New York have been used for many years and have given good satisfaction.

Specifications for Asphalt Roofing.—The following specification was prepared by the above-named company. The manner of laying the felting differs from that ordinarily employed for coal-tar roofing:

*Specifications.**—Cover the roof with two thicknesses of Warren's Composite Roofing Felt, manilla-paper side down, lapping each sheet 17 ins. over the preceding one, and securing with nails through tin discs about 2½ ft. apart. Over the entire surface of the composite felt thus laid mop an even coating of Warren's Anchor Brand Natural Asphalt Roofing Cement. Over this coating of cement lay one thickness of Anchor-brand asphalt felt, lapping each sheet at least 2 ins. over the preceding one, sticking these laps thoroughly with the hot asphalt roofing cement, and securing with nails through tin discs. Over this first sheet of Anchor-brand felt mop again an even coating of cement, and over this lay a second sheet of Anchor-brand felt, having the laps come in the middle of the first sheet of Anchor-brand felt beneath, sticking the 2-in. laps as before, and securing with nails through tin discs about 1½ ins. from the upper edge of the sheet. Over the entire surface of the felt thus laid spread an even coating of the Anchor-brand cement, covering it immediately with a sufficient body of well-screened dry gravel. If the roofing is applied in cold weather the gravel must be heated.

Asphalt roofing costs a little more than coal-tar roofing of the same grade.

An asphalt gravel roof should not have a slope exceeding ½ in. to the foot, on account of the liability to run in hot weather.

Ready Roofing.—There are a large number of so-called "ready roofings," which are prepared by cementing together two, three, or more layers of saturated felt or felt and burlap and then coating either with a hard solution of the same cementing material, or with hot pitch or asphalt into which is embedded sand or fine gravel.

These roofings are commonly put up in rolls 36 ins. wide and

* The Editor has been notified by the Warren Chemical and Manufacturing Company that their material will endure at a pitch of five to seven inches to the foot, if put in according to their directions.

are applied by lapping the strips 2 ins. with a coat of cementing material between, and nailing every 2 or 3 ins. with roofing nails with tin caps. A sufficient quantity of cement, nails, and tin caps is packed in the center of the rolls.

The particular advantage of these roofings is that no previous experience is required for laying them and no kettles are required; for this reason they are extensively used in the country, and on railroad shops, factories, and mill buildings. In cities there is no particular advantage in using them except for roofs that are too steep for coal-tar pitch, as they cost on the roof about the same as good gravel roofing.

Many of these ready roofings are as durable under ordinary conditions as the light-weight gravel roofs. In Colorado, however, it has been found that they are badly damaged by severe hail-storms, probably owing to the lack of the protecting gravel.

For roofs having a rise of 1 inch or more to the foot, these roofings make an economical and durable roof, and for some buildings are to be preferred to other materials.

The best known and most extensively used of these ready roofings are:

Brand.	Name of Manufacturer.
The P. & B. Ruberoid Roofing.....	Standard Paint Co., N. Y.
Malthoid......................	Paraffine Paint Co., San Francisco.
Arrow Brands, Asphalt Roofing....	Asphalt Ready Roofing Co., N. Y.
Elaterite Roofing................	Western Elaterite R'f'g Co., N. Y.
Elaterite Roofing................	Elaterite R'f'g Co., San Francisco.
Standard Asbestos Roofing........	H. W. Johns-Manville Co., N. Y.
Granite Roofing..................	Eastern Granite R'f'g Co., N. Y.
Carey's Magnesia Flexible Roofing..	Philip Carey M'f'g Co., Lockland, O.
Asphalt Sand-surfaced Roofing......	Warren Chemical & M'f'g Co., N. Y.

Corrugated Iron and Steel Sheets.

Corrugated sheets of iron and steel are very extensively used for the roofing and siding of mills, sheds, grain-elevators, and warehouses.

The best grades of corrugated sheets are now made of double-refined box-annealed iron or steel.* The corrugations are usually made lengthwise of the sheet, either by passing them through rolls or by pressing the plain sheets in a press made to give the desired corrugation. It is claimed that the latter

* It is claimed that "the life of a genuine *puddled-iron* sheet when exposed only to the pure air and natural elements is from five to eight times longer, and when exposed to sulphurous and other gases ten to twenty times longer, than that of steel or semi-steel of the same gauge, or a light gauge of sheet made from pure puddled pig iron will wear longer than the heaviest gauges of steel sheets, or than galvanized sheets of the same gauge."

method gives the more perfect and uniform corrugations. The weight and thickness of the metal is represented by the gauge number of the black sheets from which the corrugated sheets are made. The standard gauge for sheet iron and steel in this country is that established by act of Congress March 3, 1893.

The following table gives the weight and thickness of the different gauges, from Nos. 7 to 30, for flat *black sheets.* [The gauge extends from No. 7–0, ½ in. thick, up to No. 40, .005469 in. in thickness, but sheet steel is not commonly made thinner than No. 30, and above 3/16 in. the thickness is generally designated by fractions of an inch.]

U. S. STANDARD GAUGE FOR SHEET IRON AND STEEL.

No. of Gauge.	Thickness.		Weight.	
	Approximate Thickness in Fractions of an Inch.	Approximate Thickness in Decimal Parts of an Inch.	Weight per Square Foot in Ounces Avoirdupois.	Weight per Square Foot in Pounds Avoirdupois.
7	3/16	.1875	120	7.5
8	11/64	.171875	110	6.875
9	5/32	.15625	100	6.25
10	9/64	.140625	90	5.625
11	1/8	.125	80	5.
12	7/64	.109375	70	4.375
13	3/32	.09375	60	3.75
14	5/64	.078125	50	3.125
15	9/128	.0703125	45	2.8125
16	1/16	.0625	40	2.5
17	9/160	.05625	36	2.25
18	1/20	.05	32	2.
19	7/160	.04375	28	1.75
20	3/80	.0375	24	1.50
21	11/320	.034375	22	1.375
22	1/32	.03125	20	1.25
23	9/320	.028125	18	1.125
24	1/40	.025	16	1.
25	7/320	.021875	14	.875
26	3/160	.01875	12	.75
27	11/640	.0171875	11	.6875
28	1/64	.015625	10	.625
29	9/640	.0140625	9	.5625
30	1/80	.0125	8	.5

Section 3 of the act of Congress provides that in the practical use and application of the above gauge a variation of 2½ per cent. either way may be allowed.

Galvanizing the sheets adds approximately $2\frac{1}{2}$ ounces per square foot to the above weights.

The regular sizes of the corrugations are $2\frac{1}{2}$, $1\frac{1}{4}$, $\frac{5}{8}$, and $\frac{3}{16}$ inch, measured from centre to centre.

Besides these sizes, 5-in., 3-in., and 2-in. corrugations are made by one or two corrugating companies.

Corrugated sheets are carried in stock in 6-, 7-, 8-, 9-, and 10-ft. lengths. The 8-ft. length, however, is most commonly used. The width of the sheets, as a rule, is 24 ins. between centres of the outer corrugations, so that the covering width is 24 ins. when one corrugate is used for side lap. This applies to all sizes of corrugations, although one or two mills make wider sheets.

The 2-, $2\frac{1}{2}$-, and 3-in corrugated sheets are made in all gauges from 16 to 28, the $1\frac{1}{4}$-in. corrugated sheets are made from Nos. 22 to 28 gauge, the $\frac{5}{8}$-in corrugated sheets from Nos. 24 to 28, and the $\frac{3}{16}$-in. corrugated sheets of Nos. 26, 27, and 28 gauges only. No. 28 gauge is most used for all purposes. The sheets are generally painted with a red mineral paint before shipping; galvanized sheets can also be obtained if desired.

All corrugated sheets are sold by the square (100 sq. ft.), measuring the actual width and length of the corrugated sheets.

Corrugated Roofing.*

For covering roofs, either 3-, $2\frac{1}{2}$-, or 2-in. corrugates should be used, the $2\frac{1}{2}$-in. being the most common size. The thickness or gauge will depend on the distance between the supports on which the sheets are laid.

Nos. 26 to 28 gauges should be laid on close sheathing, or strips not more than 1 to 2 ft. between centres. The maximum distances between supports for other gauges should be as follows:†

For No. 24 gauge, 2 to $2\frac{1}{2}$ ft. from centre to centre.

For Nos. 22 and 20 gauge, 2 to 3 ft. from centre to centre.

For No. 18 gauge, 4 to 5 ft. from centre to centre.

For No. 16 gauge, 5 to 6 ft. from centre to centre.

The least pitch which should be given to roofs that are to be covered with corrugated sheets is 3 ins. to the foot, and for truss

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* Much practical information regarding the use of corrugated sheets on mill buildings, with many details, is contained in Steel Mill Buildings, by Milo S. Ketchum, C.E.

† For strength of corrugated sheets see Steel Mill Buildings, p. 190.

roofs it is not desirable to have less than a one fourth pitch (6 ins. to the foot).

When laid on a roof, corrugated sheets should have a lap at the lower end of from 3 to 6 ins., according to the pitch of the roof. For a $\frac{1}{3}$ pitch, a 3-in. lap; for a $\frac{1}{4}$ pitch, a 4-in. lap; and for a $\frac{1}{8}$ pitch, a 5-in. lap. For the side lap it is recommended that each alternate sheet be laid upside down and lapped as shown in Fig. 1. By this method, when water is blown through

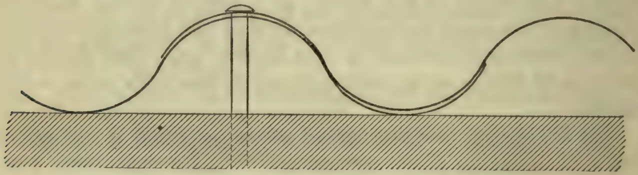

Fig. 1

the first lap, it will stop and not pass the half lap, but run down and out at the end of the sheet. A great deal of roofing, however, is laid as in Fig. 2.

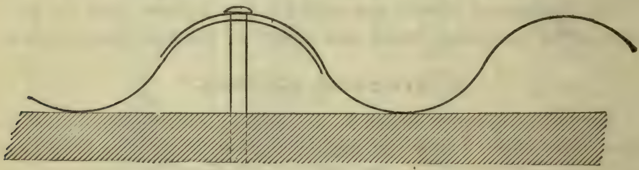

Fig. 2

In applying to sheathing or wood strips, the sheets are secured by nailing through the tops of the corrugations, the nails being driven through every alternate corrugation at the ends, and about 8 ins. apart at the sides.

When applied to iron or steel purlins, the side laps should be at least $1\frac{1}{2}$ corrugations, and the sheets should be riveted together every 8 ins. on the sides and at every alternate corrugation at the ends. The Cincinnati Corrugating Company makes a patent edge corrugation which makes a tight joint with a lap of only one corrugation. To fasten the sheets to the purlins, which are usually of angles, a cleat of band iron $\frac{3}{4}$ or $\frac{7}{8}$ of an inch wide may be passed around or under the purlins and riveted at both ends to the sheet, as shown in Fig. 3. By contracting or pressing this cleat toward the web a tight, secure fastening

is made, which allows for contraction and expansion of the sheets.

Cleats, however, are generally used only with channel or Z-bar purlins. For angle-iron purlins, the clinch-nail (of soft iron wire) is most commonly used, as shown by Fig. 4; it makes a very satisfactory fastening:

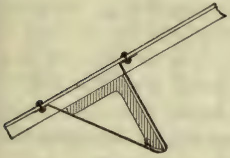

Fig. 3

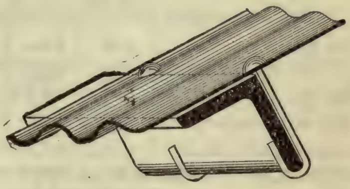

Fig. 4

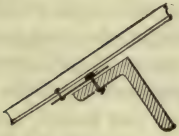

Fig. 5

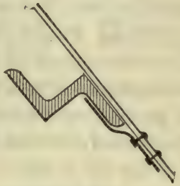

Fig. 6

The following table shows the size of clinch-nails to be used with different sizes of angle purlins and also the number of nails to the pound in each instance:

Purlin angle	2×2 ins.	$2\frac{1}{2}\times3$ ins.	$3\frac{1}{2}\times3\frac{1}{2}$ ins.	$4\times4\frac{1}{2}$ ins.
Length of nail	4 ins.	5 ins.	6 ins.	7 ins.
No. of nails per lb.	48	38	33	27

The nails should be placed through the *top* of every second or third corrugation.

At the eaves of the building and along the edge of the ventilator especial pains should be taken in fastening the roofing, as this is where the wind catches it and strips it from the purlins. For these places the best method of fastening is shown by Fig. 5.

This consists of a strip of sheet iron about 2 inches wider than the purlins, made of No. 12 iron, riveted to the purlins with $\frac{1}{4}$-in. rivets spaced 10 ins. apart; to this strip the corrugated sheets are riveted, at spaces of 5 ins. or two corrugates, with six-pound rivets. The method of fastening shown by Fig. 6 also answers very well and is less expensive.

In ordering corrugated sheets an allowance must be made for the laps. The following table gives the number of square feet necessary to cover one square of actual surface, using sheets 8 ft. long. If shorter sheets are used, the allowance must be slightly increased:

NUMBER OF SQUARE FEET OF CORRUGATED SHEETS TO COVER 100 SQ. FT. OF ROOF.

End laps.	1 in.	2 ins.	3 ins.	4 ins.	5 ins.	6 ins.
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
Side lap, 1 corrugation.	110	111	112	113	114	115
" " $1\frac{1}{2}$ "	116	117	118	119	120	121
" " 2 "	123	124	125	126	127	128

APPROXIMATE WEIGHT IN POUNDS OF 100 SQ. FT. OF $2\frac{1}{2}$-INCH CORRUGATED SHEETS.

Gauge.	No. 28	No. 27	No. 26	No. 24	No. 22	No. 20	No. 18	No. 16
Painted.	69	77	84	111	138	165	220	275
Galvanized.	86	93	99	127	154	182	236	291

Anti-condensation Lining.—Wherever corrugated steel is laid on purlins with no sheathing or paper underneath, if the building is heated, moisture will invariably collect on the under side, and if the air in the building is warm and humid, considerable dripping will result. To prevent this dripping, it is necessary to protect the under side of the corrugated steel with paper or felt. This may be done by first stretching poultry-netting over the purlins, from eaves to ridge, and wiring the strips together at the edges. Over this should be laid one thickness of asbestos paper and one or two layers of saturated felt. The corrugated steel may then be fastened to the purlins in the usual way. The side laps may be secured by stove-bolts, with $1'' \times \frac{1}{8}'' \times 4''$ plate washers on the under side, to support the lining.

Corrugated Siding.

For siding, either the $2\frac{1}{2}$-, 2-, or $1\frac{1}{4}$-in. size corrugations are used. The $1\frac{1}{4}$-in. size, however, makes the best appearance. For the lap, one inch at the bottom and one corrugation at the sides is sufficient.

For sheds, etc., the sheets may be nailed to cross-pieces cut in between the studs horizontally and spaced from 2 to 3 ft. apart, the studs being from 3 to 4 ft. on centres. For elevators, either cross-corrugated sheets or sheets not more than 32 ins. long should be used. The nails should be driven in the trough of each alter-

nate corrugation 2 ins. above the lower end of the sheet, which will be 1 in. *above* the top end of the under sheet. This will allow the sheet to slide 1 in. in 32 ins. as the building settles before the nail will strike the upper end of the lower sheet. The side lap should not be nailed.

Ceilings.—For the ceilings of stores, stables, etc., $\frac{3}{16}$-in. or $\frac{5}{8}$-in. corrugated sheets are much used; they make an excellent material for this purpose.

The cost of corrugated sheets over sheathing is about $3.50 a square and on steel purlins $4 to $4.50.

Galvanized Iron.—This term is commonly applied to all galvanized sheet metal, although most, if not all, of the galvanized sheets of the present day have a steel base.

The best quality of galvanized iron bears the trade-mark "Appollo" or "Apollo Best Bloom."

Galvanized sheets come in lengths of 6, 7, and 8 ft. in U. S. Gauge Nos. 14, 16, 18, 20, 22, 24, 26, 27, 28, and 30, and in widths of 24, 26, 28, 30, and 36 ins. for all gauges except No. 30, which is made only in widths of 24, 26, and 28 ins.

Sheets of No. 28 gauge are also made in widths of 32 and 34 ins. The widths commonly carried in stock are 24, 28, and 30 ins.

Most of the galvanized iron used for cornices and ornamental work is No. 27 gauge. No. 28 is sometimes used for gutters and conductors.

Cost.—The net price *per* 100 *lbs.* of flat galvanized sheets, in car-load lots at Pittsburg, June, 1904, is as follows: For Nos. 10 to 14, $2.35; Nos. 15 and 16, $2.50; Nos. 17 to 21, $2.60; Nos. 22 to 24, $2.75; Nos. 25 and 26, $3.00; No. 27, $3.25; No. 28, $3.50. The retail price in cities varies from 3 cts. to 4½ cts. per lb., depending largely upon the freight rate.

FLOOR AND WALL TILING.

Tile floors are extensively used in the better class of buildings, and particularly in those portions which are used by the public, on account of their great durability, sanitary qualities, and decorative effects. As a matter of fact, a good tile floor is also cheaper in the long run than a wooden floor if it is subject to much wear.

The materials used for floors are tiles made from different

grades of clay, marbles, slate, glass, and rubber. Of these the most durable and sanitary are the vitreous clay tile.

For walls and wainscotings, glazed tiles, marbles, and glass are extensively used.

Clay Tiles.—The several grades of clay tile are known under the following terms:

A. Floor Tile.

1. *Common Encaustic Tile.*—The cheapest grade, made of naturally colored clays—red, buff, gray, chocolate, and black. This tile is of a porous, absorbing character and is used for common floors of no sanitary requirements.

2. *Semi-vitreous Tile.*—A somewhat better grade of the former article, having less porosity and absorption.

3. *Vitreous Tile.*—The hardest tile known (cannot be scratched by steel or sand), non-absorbent and thoroughly aseptic. It is principally in use for floors requiring a perfect sanitary condition; is manufactured in white, blue, gray, green, and pink colors of great delicacy.

4. *"Ceramic" Tile, or Ceramic Roman Mosaic.*—This material is made of *vitreous* clay in tesseral pieces representing the tesseræ of the Roman mosaic. It is made in regular tile ranging from ½-in. to ¾-in. squares and also in hexagonal shapes from ¾ in. to 1 in. in size. A round "lozenge" is also manufactured to be laid in tesseral paving.

The material itself is of great hardness and is well suited for work of monumental or public character. The even and regular texture of the tesseræ admits the adoption of damask designs which have become identified and associated with this material. The minuteness of the tesseræ admits great range in designing and can therefore follow each line of architecture. The ceramic Roman mosaic is much preferred to mosaic consisting of natural marbles on account of the great variety in colors and also on account of its greater durability, the vitreous clay tile being perfectly impervious to attacks of any acids contained in the atmosphere, while marble especially is subject to rapid disintegration caused by the sulphuric acid contained in the smoke-laden atmosphere of our cities.

5. *Florentine Mosaic and Flint Tile.*—This is the largest and heaviest tile manufactured in this country. It is either plain or inlaid and is in use especially in ecclesiastic work on account of its relation to mediæval application. The material is vitreous

annealed and is more tough than brittle. It is also in use for exterior polychrome work.

6. *Aseptic Tile.*—A large and heavy thoroughly vitreous tile for institute work. It is the only vitreous tile of large size made in this country. As the tile is large and generally of hexagonal shape, the joint space is reduced to a minimum, and it is, therefore, especially adapted for hospitals, operating-rooms, and contagious wards in public institutions.

B. Enamelled or Wall and Mantel Tile.

1. *White Wall Tile.*—A glazed tile for wainscots. This tile has a white soft body and its surface is covered with a clear glaze. The brilliancy of this glaze and its reflecting properties makes the white wall tile especially desirable for dark passages.

2. *Colored Glaze or Enamel Tile.*—This tile is about the same as the former in quality; the "glaze," or "enamel," however, is stained with metallic oxides, which produces a brilliant decorative effect.

3. *Dull Satin, etc., Finished Enamelled Tile.*—A glazed tile with a "dull" or "blind" enamel. The dull finish is either produced by sandblasting or devitrifying enamels. It is principally used for quaint decorative effects in mantel work.

4. *Glazed Roman Mosaic.*—The latest style of enamelled tiling which has great decorative possibilities. It has the same tesseral texture as the ceramic floor tile and finds ready application to wainscots and mantel work.

Clay tile are set in Portland-cement mortar as a rule, and floors should always be provided with a substantial concrete. A new invention which has been placed on the market as "Plicaro" mosaic consists of the ceramic mosaic laid on a flexible base. With this material wood floors can be provided with tile floors, and owing to the elasticity and lightness of the material, floors in elevators, boats, and other ambulant structures can be safely tiled.

Marble Tiles from 9 to 12 ins. square have been extensively used for floors, principally on account of their decorative effect. None of the marbles, however, are as hard and consequently as durable as the vitreous and ceramic tile, and from all practical standpoints do not make as good a floor.

When used, they should be 1¼ ins. thick and not over 12 ins. square, and should be bedded in cement on a concrete base. Marbles should not be used for floors in hospitals, as they yield rapidly to the usual antiseptic floor washes.

Slate, although non-absorbent and not affected even by dilute mineral acids, is too cold and dingy to commend itself as a floor tile, but because it is conveniently handled in large slabs it is valuable as a cheap base and as a cover for wiring and pipe trenches in the floor. As these often follow a wall, it may serve in the capacity of a border and as such be extended around the floor space. Slate slabs for floors should be about $1\frac{1}{4}$ ins. thick.

Marbleithic Tile or Slabs are made of small pieces or chips of marbles of irregular shapes, set in a backing of sand and Portland cement, and after the cement has set, the top surface is rubbed until it becomes flat and smooth. Marbleithic resembles mosaic or Terazzo, except that it is laid in the form of tiles instead of being put down on the floor in a plastic condition. Much objection has been made to Terazzo because of the cracks which commonly occur in it, due to the slight settlements which are unavoidable in a new building. With tile floors of any material the joints allow for any slight movement of the floor, without producing visible cracks. By the process of manufacture, marbleithic is made much harder than it is possible to make mosaic floors that are laid in a plastic condition, so that they have a much better wearing surface. Floors of this material have now (1904) been in use for nine years and they have been found to show but little if any wear.

Marbleithic tiles are made of various colored marbles and in different sizes, shapes, and patterns, so that a great variety of effects may be produced.

Sanitary coved base, stair treads, and wainscoting are also made of it.

Cast Glass Tile, while quite resistant to a blow when the polish is unbroken, will break very easily when the surface is scratched. All glass tile should, therefore, be very thick and small or protected by metal framing.

Novus Sanitary Glass * is a sanitary structural glass manufactured in all thicknesses from $\frac{1}{2}$ in. up to 2 ins. and in slabs of all widths and lengths up to 100 ins. wide and 180 ins. long. It is made in various colors and designs and in the following finishes: natural fire finish, hone, semi-polished, and polished.

This material can be worked and handled the same as marble, it is readily drilled and shaped to accommodate fixtures, etc., and is very handsome in appearance. It is impervious to discoloration and is non-crazing.

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* Made by the Penn-American Plate Glass Company, Pittsburg, Pa.

These qualities make it especially desirable for floors, wainscoting, tables, shelves, etc., in all places where an absolute sanitary condition is desired, combined with a handsome appearance.

Interlocking Rubber Tiling.

Several years ago the New York Belting and Packing Company introduced an interlocking rubber tile, which, because of its being noiseless, non-slippery, and more comfortable to the feet than inelastic substances, has met with great favor for floors in banking-rooms, counting-rooms, vestibules, elevators, stairs, cafés, libraries, churches, etc. For elevators it is the most durable and practical floor that can be laid; it is also especially and peculiarly adapted for floors of yachts and steamships. The interlocking feature unites the tiles into a smooth, unbroken sheet of rubber, unlimited in area. The tiles do not pull apart or come up, and each being distinct, almost any color scheme can be secured, the tiles being made in a carefully selected variety of colors. The tiles are laid directly over the original floor, like a carpet (except that they are not fastened). Experience has shown that they are very durable.

Each tile is $2\frac{3}{8}$ ins. square and $\frac{3}{8}$ in. thick; 25.5 tiles are required to the square foot. Rubber nosing for stairs is made to interlock with the tile.

Cost of Different Tiles.

The following prices are approximately the cost (to the trade) at the factory at the present time (1904). To this should be added freight and the dealers' profit. The cost of laying the tiles on a cement base (in addition to the cost of the tiles) should not exceed 25 cents per square foot.

FLOOR TILES.

	Factory Price per Sq. Ft.	
Common encaustic tile, unglazed	15	cts.
Vitreous tile: white	22½₀	"
Colors (large sizes)	from 23 to 26	"
"Ceramic" tile, or ceramic Roman mosaic,	from 20 to 35	"

WALL AND MANTEL TILE.

	Factory Price per Sq. Ft.	
White glazed wall tile	25	cts.
Colored glaze or enamel tile	30	"
Enamel tile, dull satin finish	40	"

Marbleithic costs from 45 cts. per sq. ft., upwards, laid.

ASPHALTUM.

"*Bitumen, Asphaltum, Asphalt.*—Bitumen is the name used to denote a group of mineral substances, composed of different hydrocarbons, found widely diffused throughout the world in a variety of forms which grade from thin volatile liquids to thick semi-fluids and solids, sometimes in a free or pure state, but more frequently intermixed with or saturating different kinds of inorganic or organic matter.

"To designate the condition under which bitumen is found, different names are employed; thus the liquid varieties are known as naphtha and petroleum, the semi-fluid or viscous as maltha or mineral tar, and the solid or compact as asphaltum or asphalt." *

Asphaltum is found in extensive beds or lake-like deposits on both continents; the most notable of these are the pitch lakes on the island of Trinidad, and at Bermundez, Venezuela.

It is also found saturating the limestone and sandstone formations in certain localities.

Deposits of very nearly pure asphaltum are found in Utah, Mexico, Cuba, and various parts of the United States.

Elaterite, gilsonite, and wurtzilite are varieties of very nearly pure asphaltum.

Asphaltic roofing materials are manufactured principally from Trinidad asphalt. These deposits have also been the main source of supply for the asphaltum used in street-paving in the United States.

The term **rock asphalt** is commonly used to designate the material obtained from the bituminous limestone deposits at Seyssel and Pyrimont, in the valley of the Rhone, France, and in the Val-de-Travers, canton of Neuchâtel, Switzerland, and at Ragusa, on the island of Sicily. It is extensively employed for paving purposes throughout Europe, and is considered to make a much more durable pavement than can be made with asphaltum.

Rock asphalt is prepared for shipment in two forms: (*a*) *compressed asphalt blocks*, which are used for paving in much the same way as stone blocks, and (*b*) *mastic asphalt*, which is put up in cakes of varying shape, generally bearing the manufacturer's trade-mark.

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* Byrne, Inspectors' Pocket Book.

In the Eastern States *mastic asphalt* is used for floors of cellars, stores, breweries, malt-houses, hotel kitchens, stables, laundries, conservatories, public buildings, carriage-factories sugar-refineries, mills, rinks, etc., and for any place where a hard, smooth, clean, dry, fire- and water-proof, odorless, and durable covering of a light color is required, either in basement or upper stories. It can be laid either over cement concrete, brick, or wood in one sheet without seams; also over cement concrete for roofs for fire proof buildings. For dwelling-house cellars, especially on moist or filled land, this material is especially adapted, being water-tight, non-absorbent, free from mould or dust, impervious to sewer-gases, and for sanitary purposes invaluable.

Mastic asphalt is also valuable for *damp courses* over foundations, and for covering vaults and arches under ground.

For floors of cellars, courtyards, etc., laid on the ground, a base of cement concrete 3 ins. thick should first be laid; and over this a layer of asphalt from $\frac{3}{4}$ in. to $1\frac{1}{2}$ ins. thick, according to the use to which it is to be put. For ordinary cellar floors, the asphalt need not be more than $\frac{3}{4}$ in. thick; for yards on which heavy teams are to drive, it should be $1\frac{1}{2}$ ins. thick. In specifying asphalt pavement, both the thickness of the concrete and of the asphalt should be given; it should also be remembered that "asphalt pavement" does not include the concrete foundation unless so specified.

In laying asphalt over plank or boards, a layer of stout, *dry* (not tarred) sheathing-paper should first be put down and the asphalt laid on this. Asphalt floors for stables should be at least 1 in. thick. The *cost* of rock asphalt in the large cities varies from 12 to 17 cents per square foot in jobs of 2,000 feet and over; this does not include the concrete foundation. German and other cheap asphalts are laid for somewhat less, while imitation rock asphalts are furnished for considerably less.

Architects and owners desiring to employ *rock asphalt* for any of the above purposes should be careful to secure the genuine *Val-de-Travers* or *Seyssel* or *Sicilian* rock asphalt, as there are imitations which are of but little value.

The *bituminous sandstones* of California have been extensively used for paving streets in Western cities. They are prepared for use as a paving material by crushing to powder. With this powder a considerable proportion of sand or gravel is generally mixed and the mixture is then heated until it becomes

plastic and then spread upon the street and compressed by rolling.

MINERAL WOOL.*

There are at least two kinds of mineral wool made in this country. The more common kind is made by converting the slag † of blast-furnaces, mixed with certain rocks while in a melted condition, to a fibrous state.

Its appearance is much like that of wool, being soft and fibrous, but in no other respect are the materials alike. Mineral wool made from slag appears in a variety of colors, principally white, but often yellow or gray and occasionally quite dark. The color, however, is said to be no indication of the quality, as all of the peculiar properties of the material are present in equal proportions in any of the shades. The other kind of mineral wool is known as *rock wool*, and is made from granite rock raised to 3,000 degrees of temperature. It is claimed to be absolutely free from sulphur and the only odorless wool manufactured; it has been approved by the U. S. War Department. Its color is white and its general apparance is the same as that made from slag. The peculiar nature of both kinds is that of a mass of very fine, pliant, but inelastic, vitreous fibres interlacing each other in every direction and forming an innumerable number of minute air-cells. Its great value in the insulation and protection of buildings lies in the number of air-cells which it contains, combined with its resistance to heat or fire. In common slag wool 92 per cent. of the volume consists of air held in minute cells, while in the best grade the proportion of air reaches as high as 96 per cent. This confined air makes it one of the best, if not the best, of the non-conductors of heat, and to a less degree of sound. Aside from these qualities it is very durable, contains nothing that can decay or become musty, and is almost a sure protection against rats and vermin.

Ordinary mineral wool weighs about 12 pounds per cubic foot, and is put up in bags containing from 40 to 60 pounds in each bag. It costs at the works, in Stanhope, N. J., 1 cent per pound, and at the store in New York City 1¼ cents per pound.

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* For the uses of mineral wool in building construction see Part II, Building Construction and Superintendence, p. 208.

† The best being from slag that does not contain iron.

Extra mineral wool weighs about 9 pounds per cubic foot, and is put up in bags containing from 20 to 30 lbs. in each bag. It costs at the works 4 cents per pound, and at the store, New York City, 4½ cents per pound.

In estimating the quantity of wool required for filling, 1 pound per square foot should be allowed for each inch in thickness for ordinary wool and ¾ pound for selected wool.

ESTIMATING THE COST OF STRUCTURAL STEEL FOR BUILDINGS.

Structural steel for buildings is commonly made up of I beams, channels, angles, Z bars, and plates, which may be used as single beams or braces, or built into riveted girders, columns, or trusses. The cost of the completed steel work is made up of the following items:

(1) Cost of the plain steel at the mill, plus freight and dealers' profit.

(2) Extras for cutting, punching, fitting, and assembling into girders, columns, or trusses.

(3) Cost of the fittings, such as connection angles, gusset plates, etc

(4) Shop painting.

(5) Cost of erection at the building.

(6) Painting after erection.

Base Price of Steel.—For orders of any considerable size, the cost of plain steel is based on the price at Pittsburg, plus the freight to the point of delivery.

The base price at Pittsburg at the present time (July, 1904) is \$1.60 per 100 lbs. for beams and channels 15 ins. and less, and for angles and Z's, 3 to 6 ins.

Beams and channels over 15 ins. cost 10 cts. per 100 lbs. extra, and T's over 3 ins., 5 cts. extra.

For angles, channels, and T's under 3 ins. the base is \$1.90 from Chicago stock (see page 1456).

For plates ¼ in. thick and over the base is \$1.60 per 100 lbs. For plates 3/16 in. thick, add 10 cts. per 100 lbs.

Freight Rates at present are: Pittsburg to Chicago, 16½ cts. per 100 lbs.; to St. Louis, 22 cts.; to New York, 14½ cts.; to Kansas City, 42½ cts.; to Denver, 92½ cts.; and to San Francisco, 85 cts.

For Pacific coast points, a discount of about 18 per cent. is

made from the base, at Pittsburg, on account of the high freight, and to meet European competition. On account of the expense of carrying beams in stock, local dealers usually charge from ½ to 1 ct. a pound extra on orders supplied from stock.

List of Extras to be Added to Price of Plain Beams and Channels.— If any kind of work whatever is done on the plain steel, or if the same is cut to length with a less variation than ⅜ in., an extra price is charged, which is based on the following list, adopted in 1902, and still in force. These charges are common to all shops if the order is of any size, and are not likely to be changed for some time.

In Effect July 1, 1904.

Extras to be Added to Base Price for Each 100 Lbs.

1.	For cutting to length with less variation than plus or minus ⅜ in.	15 cts.
2.	Plain punching one size hole in web only	15 "
3.	Plain punching one size hole in one or both flanges	15 "
4.	Plain punching one size hole in either web and one flange or web and both flanges	25 "
5.	Plain punching each additional size hole in either web or flanges, web and one flange, or web and both flanges	15 "
6.	Plain punching one size hole in flange and another size hole in web of the same beam or channel	40 "
7.	Punching and assembling into girders	35 "
8.	Coping, ordinary bevelling, including cutting to exact length, with or without punching; including the riveting or bolting of standard connection angles	35 "
9.	For painting or oiling, one coat, with ordinary oil or paint	10 "
10.	Cambering, beams and channels, and other shapes for ships or other purposes	25 "
11.	Bending, or other unusual work	Shop rates
12.	For fittings, whether loose or attached, such as angle connections, bolts, and separators, tie-rods, etc.	$1.55

Tie-rods in all cases, where estimated upon in connection with beams or channels, to be classified as fittings.

In making an estimate of the steel work from the framing plans, the weight of all connection angles, gusset plates, separators, tie-rods, etc., must be taken off separately, and the cost figured at $1.55 per 100 lbs. above the base price.

The weights of standard connections are given on pages 548 and 549, and of standard separators on page 544.

In estimating cost of riveted columns and girders, the weight of the plain bars and plates of which the column or girder is composed may be taken, and an extra added to the price per pound to cover cost of rivets and assembling.

This extra will be about as follows:

Light channel or Z-bar columns.	$1\frac{1}{2}$	cts.	per	lb.
Heavy channel or Z-bar columns.	$1\frac{1}{4}$	"	"	"
Plate girders, 24 to 48 ins. deep.	$1\frac{1}{4}$	"	"	"
Box girders, 24 to 48 ins. deep.	$1\frac{1}{4}$	"	"	"
Box girders, 48 to 60 ins. deep.	$1\frac{1}{10}$	"	"	"

Cost of Erecting.—For erecting ordinary beams and columns in buildings having masonry walls the cost of erection should not exceed $10 per ton with bolted connections, and will sometimes be as low as $6 per ton.

For erecting the steel work of skeleton buildings having riveted connections, it is common to allow $10 per ton.

Cost of Painting.—The common charge for shop painting is $1 per ton, but if done in accordance with the specification on page 1413 it would exceed this amount.

For painting one additional coat after erection, allow $2 per ton.

Roof-trusses.—In lots of at least six, the shop cost of ordinary roof-trusses in which the ends of the members are cut off at right angles was about as follows in 1902:* Trusses weighing 1,000 lbs. each, $1.15 to $1.25 per 100 lbs.; trusses weighing 1,500 lbs. each, $0.90 to $1.00 per 100 lbs.; trusses weighing 2,500 lbs. each, $0.75 to $0.85 per 100 lbs.; and trusses weighing 3,500 to 7,500 lbs., $0.60 to $0.75 per 100 lbs. Pin-connected trusses cost from 10 to 20 cts. per 100 lbs. more than riveted trusses. (M. S. Ketchum, C.E. in Steel-Mill Buildings.)

Steel-mill Buildings.—The average shop cost for the frame of steel-mill buildings, including draughting is about $25.00 per ton, and the cost of erection from $15.00 to $25.00 per ton.

(A great amount of data pertaining to the cost of steel-mill buildings is given by Mr. Ketchum in the book above mentioned.)

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* Under present conditions, July 1, 1904, these figures should be increased 25 per cent.

Cost of Drafting.—Details for church and court-house roofs having hips and valleys cost from $6.00 to $8.00 per ton; details for ordinary mill buildings cost from $2.00 to $4.00 per ton. The details for all work fabricated by the Gillete-Herzog Mfg. Co., with the exception of plain beams and complicated tank-work, were made in 1896 by contract, by Mr. H. A. Fitch, now structural engineer for the Minneapolis Steel and Machinery Co., Minneapolis, for $2.60 per ton. This price netted the contractor a fair profit.*

Approximate Estimates of the Weight of Steel in Buildings.—According to H. G. Tyrrell, C.E.,† the weight of steel in any proposed new building may be roughly estimated by the following data, which is a fair average for buildings not over eleven stories high, designed according to the Building Laws of the City of Boston:

	Per Sq. Ft. of Floor.
Apartment houses and hotels, with outside frame	14 lbs.
Apartment houses without outside frame	9 "
Office buildings, with outside frame	23 "
Office buildings, without outside frame	15 "
Warehouses, with outside frame	28 "
Warehouses, without outside frame	18 "

For buildings higher than eleven stories, the weight of floors will increase in direct proportion to the number of stories, while the weight of columns will increase more rapidly.

For the approximate weight of roof-trusses, see pages 947 and 949.

Cost of Merchant Steel.—The cost of merchant iron and steel of all kinds is based on a certain size of each particular shape, which is taken as the "base," and the price of all other sizes is figured at a certain extra above the base. The base price may fluctuate and be changed without notice, but the extras remain constant, and are the same in all localities. Following is the

* M. S. Ketchum.

† Estimating Structural Steel, in Architects & Builders' Magazine, Jan., 1903.

STANDARD STEEL CLASSIFICATION

In Effect July 1, 1904.

ROUNDS AND SQUARES.

Size	Extra per 100 lbs.	Size	Extra per 100 lbs.
3/4 to 3 ins.	Base	3 1/16 to 3 1/2 ins	0.15
5/8 to 11/16 in.	$0.10	3 9/16 to 4 ins	0.25
1/2 to 9/16 in.	0.20	4 1/16 to 4 1/2 ins	0.30
7/16 in.	0.40	4 9/16 to 5 ins.	0.40
3/8 in.	0.50	5 1/8 to 5 1/2 ins.	0.50
5/16 in.	0.60	5 5/8 to 6 ins.	0.75
1/4 and 9/32 in.	0.70	6 1/8 to 6 1/2 ins.	1.00
1/32 in.	1.00	6 5/8 to 7 1/4 ins.	1.25
3/16 in.	2.00		

FLAT BARS AND HEAVY BANDS.

Width	Thickness	Extra
1 to 6 ins.	× 3/8 to 1 in.	Base
1 to 6 ins.	× 1/4 and 5/16 in.	$0.20 extra per 100 lbs.
11/16 to 15/16 in.	× 3/8 to 3/4 in.	0.40 " " " "
11/16 to 15/16 in.	× 1/4 and 5/16 in.	0.50 " " " "
9/16 and 5/8 in.	× 3/8 to 1/2 in.	0.50 " " " "
9/16 and 5/8 in.	× 1/4 and 5/16 in.	0.70 " " " "
1/2 in.	× 3/8 and 7/16 in.	0.90 " " " "
1/2 in.	× 1/4 and 5/16 in.	1.10 " " " "
7/16 in.	× 3/8 in.	1.00 " " " "
7/16 in.	× 1/4 and 5/16 in.	1.20 " " " "
3/8 in.	× 1/4 and 5/16 in.	1.50 " " " "
1 1/8 to 6 ins.	× 1 1/16 to 1 3/16 ins	0.10 " " " "
1 1/8 to 6 ins.	× 1 1/4 to 1 1/2 ins.	0.20 " " " "
1 3/4 to 6 ins.	× 1 5/8 to 2 3/4 ins.	0.30 " " " "
3 1/8 to 6 ins.	× 3 to 4 ins.	0.40 " " " "

LIGHT BARS AND BANDS.

Width	Thickness	Extra
1 1/2 to 6 in.	× Nos. 7, 8, 9 and 3/16 in.	$0.40 extra per 100 lbs.
1 1/2 to 6 in.	× Nos. 10, 11, 12 and 1/8 in.	0.60 " " " "
1 to 1 7/16 in.	× Nos. 7, 8, 9 and 3/16 in.	0.50 " " " "
1 to 1 7/16 in.	× Nos. 10, 11, 12 and 1/8 in.	0.70 " " " "
13/16 to 15/16 in.	× Nos. 7, 8, 9 and 3/16 in.	0.70 " " " "
13/16 to 15/16 in.	× Nos. 10, 11, 12 and 1/8 in.	0.80 " " " "
11/16 and 3/4 in.	× Nos. 7, 8, 9 and 3/16 in.	1.00 " " " "
11/16 and 3/4 in.	× Nos. 10, 11, 12, and 1/8 in.	1.20 " " " "
9/16 and 5/8 in.	× Nos. 7, 8, 9 and 3/16 in.	1.20 " " " "
9/16 and 5/8 in.	× Nos. 10, 11, 12 and 1/8 in.	1.30 " " " "
1/2 in.	× Nos. 7, 8, 9 and 3/16 in.	1.30 " " " "
1/2 in.	× Nos. 10, 11, 12 and 1/8 in.	1.50 " " " "
7/16 in.	× Nos. 7, 8, 9 and 3/16 in.	1.80 " " " "
7/16 in.	× Nos. 10, 11. 12 and 1/8 in.	2.10 " " " "
3/8 in.	× Nos. 7, 8, 9 and 3/16 in.	1.90 " " " "
3/8 in.	× Nos. 10, 11, 12 and 1/8 in.	2.40 " " " "

For intermediate sizes, the next higher extra to be charged in all cases.

Angles.

$1\frac{1}{2}\times\frac{3}{16}$ ins. and heavier, but under 3 ins.	Base
1 to $1\frac{1}{4}\times\frac{3}{16}$ ins. and heavier	\$0.10 extra per 100 lbs.
$\frac{7}{8}\times\frac{3}{16}$ in.	0.20 " " " "
$\frac{3}{4}\times\frac{3}{16}$ in.	0.30 " " " "
$\frac{5}{8}\times\frac{1}{8}$ in.	2.00 " " " "
$\frac{1}{2}\times\frac{1}{8}$ in.	3.00 " " " "
3×3 ins. $\times$ less than $\frac{1}{4}$ in. thick.	0.50 " " " "
Angles $\frac{3}{4}$ in. and larger, but smaller than 3 ins. $\frac{1}{8}$ in. thick.	0.10 per 100 lbs over $\frac{3}{16}$ in.

Channels.

$1\frac{1}{2}\times\frac{3}{16}$ ins. and heavier, but under 3 ins.	Base
1 to $1\frac{1}{4}\times\frac{3}{16}$ ins. and heavier.	\$0.10 extra per 100 lbs.
$\frac{7}{8}\times\frac{3}{16}$ in.	0.20 " " " "
$\frac{5}{8}$ and $\frac{3}{4}\times\frac{3}{16}$ in	0.30 " " " "
$\frac{5}{8}\times\frac{1}{8}$ in.	0.60 " " " "
$\frac{1}{2}\times\frac{1}{8}$ in. and thicker.	1.00 " " " "
Channels $\frac{3}{4}$ in. and wider, but under 3 ins., $\frac{1}{8}$ in. thick.	0.10 per 100 lbs. over $\frac{3}{16}$ in.

Tees.

$1\frac{1}{2}\times\frac{3}{16}$ ins. and heavier, but under 3 ins.	Base
$1\frac{1}{4}\times\frac{3}{16}$ ins. and heavier.	\$0.10 extra per 100 lbs.
1 to $1\frac{1}{8}\times\frac{3}{16}$ ins. and heavier.	0.20 " " " "
$\frac{7}{8}\times\frac{1}{8}$ in. and thicker.	0.50 " " " "
$\frac{3}{4}\times\frac{1}{8}$ in. and thicker.	0.60 " " " "
$\frac{5}{8}\times\frac{1}{8}$ in. and thicker	2.00 " " " "
Tees 1 in. and larger, but smaller than 3 ins., $\frac{1}{8}$ in. thick.	0.10 cts. per 100 lbs. over $\frac{3}{16}$ in.

For intermediate sizes, the next higher extra to be charged in all cases.

The base for all of the above, from stock in Chicago, on full car-load lots, is \$1.90 (July 1, 1904). For other principal points the base may be obtained by adding the freight rates given on page 1451.

Example.—What is the probable cost at Kansas City of $1\frac{1}{4}''\times\frac{1}{4}''$ flat steel bars.

Ans.

Base	= \$1.90 per 100 lbs.
Extra	= 0.20 " " "
Freight	= $0.42\frac{1}{2}$ " " "
Total	= \$$2.52\frac{1}{2}$ per 100 lbs.

or about $2\frac{1}{2}$ cts. a pound in car-load lots. For small lots, about 3 cts. per pound might be charged.

COST OF BUILDINGS PER CUBIC FOOT.

The most accurate method of estimating the cost of any proposed building, before the plans and specifications are sufficiently complete for taking off the actual quantities, is by means of the cubic contents.

Two buildings built in the same style, and for the same purpose, of the same materials, and on the same scale of wages and and prices of materials, should cost the same, or very nearly the same, per cubic foot, although one building be somewhat larger than the other and of different shape.

It therefore follows that if we know the cost per cubic foot of different classes of buildings, in different localities, we can approximate quite closely the cost of any proposed building by multiplying its cubic contents in feet by the known cost per cubic foot of a similar building already built in that locality.

Conversely, if the cost of a proposed building must be kept absolutely within a certain sum, the size of the building should be proportioned so that the cubic contents shall not exceed the quotient obtained by dividing the amount appropriated by the average cost per cubic foot of similar buildings. Even then it may be found, when the bids are opened, that they exceed the appropriation, but the excess will probably not be so great but that the necessary reductions can be made without altering the main features of the building.

In estimating the cost by the cubic contents, it is of course necessary that the contents be computed on the same basis, in both the proposed building and the one already built. In the following examples, the cubic contents are computed from the basement or cellar floor, to the average height of a flat roof, or, if a pitch roof, the finished portion of the attic is included, or that part which might be finished, but mere air-spaces and open porches are not included. Vaults and areas under sidewalks, etc., are included as part of the basement. All measurements are to the outside of the walls and foundations. Cost does not, as a rule, include the architect's fee. A few of the examples, that were not compiled by the author, may not be computed closely by the above rule, but it is to be presumed that they are.

It should be remembered in using this table, that wages and building materials were considerably higher during the years 1901–1903 than in the years from 1893–98, thereby increasing the cost per cubic foot from 14 to 22 per cent.; also that the cost

of first-class fire-proof buildings is greater in the Western and Southern States than in the Eastern States, because of the distance from the great steel and material centres.

TABLE FOR ESTIMATING THE APPROXIMATE COST OF A NEW BUILDING, OR THE VALUE OF AN EXISTING BUILDING.

(Based on prices for labor and materials as they were in 1902.)

FARM AND COUNTRY PROPERTY.*

	Cents
Dwellings, frame: Small box house, no cornice	4
Dwellings, frame: Shingle roof, small cornice, no sash weights, plain	5 to 6
Dwellings, brick: Same class	7 to 8
Dwellings, frame: Shingle roof, good cornice, sash weights, blinds (good house)	7 to 8
Dwellings, brick: Same class (good house)	9 to 10
Barns, frame: Shingle roof, not painted, plain finish	1½ to 2½
Barns, frame: Shingle roof, painted, good foundation	2½ to 3
Stores, frame: Shingle roof, painted, plain finish	5 to 7
Stores, brick: Shingle roof, painted, good cornice, well finished	7 to 9
Ordinary wood churches and schoolhouses: country	5 to 7
Brick churches and schoolhouses: country	8 to 10

If slate or metal roof, add ¼ ct. per ft. to above.

CITY AND VILLAGE PROPERTY.*

Dwellings, frame: Shingle roof, pine floors and finish, no bathroom or furnace, plain finish (good house)	6 to 7
Dwellings, brick: Same class	8 to 9
Dwellings, frame: Shingle roof, hard-wood floor in hall and parlor, bath, furnace, and fair plumbing	8 to 9
Dwellings, brick: Same class	8 to 10
Dwellings, frame: Shingle roof, hard wood in first floor, good plumbing, furnace, artistic design, some interior ornamentation, well painted	10 to 12
Dwelling brick: With good plumbing, bath, hot and cold water, pine finish, well painted, no hard-wood finish	11 to 12

MISCELLANEOUS BUILDINGS.†

Abattoirs and other slaughter-houses	14 to 16

Asylums—lunatic—per cubic foot, complete, including patients' wards, administrative buildings, chapel, hospital, mortuary, laundry, workshops, and all other accessories, 16 to 25 cts., or from $1,350 to $1,600 per patient.

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* These figures were compiled by James N. Brown of St. Louis, Mo., and form part of instructions furnished by insurance companies to their adjusters.

† The following data is from an article by Fred T. Hodgson in the Architects and Builders' Magazine, May, 1902.

Bath-houses, complete, or for barracks, but not supplied with hot water, per cubic foot, 45 to 50 cts., or per bath, $280 to $320.

Baths, public, comprising swimming-baths, slipper-baths, laundry, caretaker's quarters, machinery, etc., complete, per cubic foot, 30 to 36 cts.

Breweries, complete, including buildings, cellarage, boilers, engine, machinery, coppers, liquor-baths, mash-tubs, coolers, refrigerator, ice storage, pumps, and all other requirements, per cubic foot, 14 to 20 cts.

Churches, plain, per cubic foot, from	16 to 22 cts.
Per square foot, from	$4.50 to $6.50
Per sitting, from	$40 to $55
Churches, ornamental, per cubic foot, from	22 to 39 cts.
Per square foot, from	$7.00 to $12.50
Per sitting, from	$65 to $120
Cotton mills, as generally constructed:	
Per cubic foot	9 to 12 cts.
Per spindle	22 to 30 cts.
Cow-stables, complete, with iron finishings and fittings:	
Per cubic foot	14 to 16 cts.
Per square foot	$2.20 to $2.80
Per cow	$170 to $190
Second-class stable with common fittings:	
Per cubic foot	11 to 13 cts.
Per square foot	$1.65 to $2.00
Per cow	$130 to $145
Third class, for farm, wood fittings:	
Per cubic foot	7½ to 10 cts.
Per square foot	$1.45 to $1.50
Per cow	$90 to $105
Drill-halls or sheds for infantry:	
Per cubic foot	11 to 14 cts.
Per square foot	$1.60 to $1.70

Electric stations of power-houses, buildings erected complete, exclusive of machinery and plant, per cubic foot, 14 to 17 cts.

Flats, as constructed in New York, comprising ornamental brickwork in front, elevators, fire-resisting floors and the whole well finished in ordinary wood throughout:

Per cubic foot	28 to 36 cts.
Hospitals, complete, including administrative buildings, etc.:	
Per cubic foot	20 to 30 cts.
Per bed	$1,550 to $2,300
Cottage hospitals for small towns:	
Per cubic foot	17 to 22 cts.
Per bed	$1,050 to $1,550
Hospitals, isolated, including all nursery buildings:	
Per cubic foot	17 to 22 cts.
Per bed	$1,800 to $2,300
Hotels, complete in every particular:	
First-class, per cubic foot	31 to 41 cts.
Second-class, per cubic foot	23 to 31 cts.
Third-class, per cubic foot	20 to 24 cts.

Houses, complete, in brickwork and good substantial finishings:

First class—Large mansion with elaborate finish:

Main building, 16-ft. ceiling, per cubic foot, 30 to 40 cts.; per square foot, $5.50 to $6.50.*

Additions, 11-ft. ceilings, per cubic foot, 16 to 20 cts.; per square foot, $2.50 to $3.00.

Second class—Large mansion of ordinary character:

Main building, 14-ft. ceiling, per cubic foot, 22 to 30 cts.; per square foot, $3.50 to $4.50.

Additions, per cubic foot, 15 to 20 cts.; per square foot, $1.65 to $2.15.

Third class—Country houses:

Height of ceiling, 11 ft., per cubic foot, 15 to 20 cts.; per square foot, $2.15 to $2.65.

Fourth class—Speculative buildings:

Ceilings, 10 ft., per cubic foot, 13 to 15 cts.; per square foot, $1.30 to $1.55.

Fifth class—Tenements and cottages to rent:

Ceilings, 9 ft., per cubic foot, 10 to 12 cts.; per square foot, $1.10 to $1.35.

Libraries, public, complete in every particular:	
Per cubic foot	16 to 22 cts.
Municipal lodging-houses for cities and large towns:	
Per cubic foot	15 to 18 cts.
Per bed	$300 to $375
Museums, public:	
For large cities, per cubic foot	22 to 33 cts.
Towns	19 to 26 cts.
Music halls, complete, per head of accommodation:	
For large cities	$80 to $130
For small cities and towns	$40 to $70
Town halls, complete:	
Large cities, per cubic foot	31 to 36 cts.
Small cities and towns	22 to 30 cts.
Alternative prices:	
Basement, per cubic foot	20 to 24 cts.
Superstructure, per cubic foot	27 to 35 cts.
Ornamental towers, per cubic foot	39 to 46 cts.
Theatres, complete, per head of accommodation:	
In large cities	$82 to $108
Small cities and towns	$50 to $80
Per cubic foot	28 to 38 cts.

Chimney shafts, plain, as for factories, etc., complete, including foundations, iron cap, etc., height measured from surface of ground to top of cap:

	Per foot in heght.
Not exceeding 100 ft. in height	$40 to $46
100 ft. to 180 ft. high	$45 to $52
180 ft. to 250 ft. high	$50 to $56

*The prices per square foot, in this and following paragraphs, are evidently per sq. ft. of floor area, counting all of the floors above the basement.—Author.

EXAMPLES OF THE ACTUAL COST OF BUILDINGS PER CUBIC FOOT.

COMPILED BY THE AUTHOR.

Office Buildings.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
Chamber of Commerce, Boston, Mass.	1891–2	Seven stories; pitch roof, iron and slate; granite walls, pile foundation; fire-proof construction; marble and oak finish.	29
"Ames Building," Boston.	1889–91	Thirteen stories; granite and Ohio stone fronts; flat roof; fire-proof construction; marble and oak finish.	53
Exchange Building, Boston.	1889–91	Nine stories; granite front; flat roof; fire-proof construction marble and oak finish.	40
United States Trust Co. Building, New York.	1888	Ten stories; flat roof; massive granite front; fire-proof construction; extra foundation; fixtures, rich marble work and finish.	60
Seven-story Office Building, New York (R. W. Gibson).	1890	Two massive stone fronts; fire-proof construction; usual machinery, fixtures, etc., complete.	37
Six-story Office Building, New York (R. W. Gibson).		Three brick and terra-cotta fronts; non-fire-proof, but with metal lathing; terra-cotta furring; machinery, elevators, etc.	26
Herald Building, New York City.	1893	Two stories and basement; tile and fire-proof roof, brick and stone fronts; fire-proof construction.	46
Auditorium Building, Chicago.	1887–9	(See description elsewhere.)	36
Rookery Building, Chicago.	1886	Eleven stories; flat roof; fire-proof construction; oak finish, marble floor and wainscot; eleven elevators.	32
Masonic Temple, Chicago.	1891	Twenty stories; pitch roof; granite and terra-cotta fronts; skeleton construction; fire-proof; rich marble and metal work; fourteen elevators.	58
Old Colony Building, Chicago.	1893–4	Seventeen stories; flat roof; Bedford stone, white brick, and terra-cotta fronts; skeleton construction; fire-proof; rich marble and metal work; six elevators.	41
N. Y. Life Insurance Building, La Salle and Monroe Streets, Chicago.	1893–4	Twelve stories; flat roof; first three stories dressed granite; terra-cotta above; riveted skeleton construction; fire-proof; machinery; rich marble work and finish; small vaults; five elevators.	47
Stock Exchange Building, La Salle and Washington Streets, Chicago.	1893–4	Thirteen stories; flat roof; skeleton construction; fire-proof; rich terra-cotta facing.	35½
Manhattan Building, Chicago.*	1892	Sixteen stories; five elevators; two fronts; pressed brick, terra-cotta, and granite.	17⅓

* Jenney and Mundie, architects; see *Inland Architect* for March, 1902.

ACTUAL COST OF BUILDINGS—(*Continued*).

Office Buildings.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
Fort Dearborn Building, Chicago.*	about 1893	Twelve stories; pressed brick and terra-cotta.	36 4/10
Isabella Building, Chicago.*	1893	Twelve stories; granite and terra-cotta; exposed on three sides; tile roof.	57¼
Board of Trade Building, Montreal, Canada.	1892–3		20
Chamber of Commerce, Cincinnati.	1887–8	Pitch roof; seven stories; granite fronts; fire-proof construction.	26
Wainwright, Building, St. Louis.	1890	Ten stories; flat roof; stone facing first and second stories; rich terra-cotta above; skeleton construction; fire-proof; four elevators.	24 9/10
Equitable Building, Denver.	1891–2	Nine stories; flat roof; granite front two stories; light brick and terra-cotta above; fire-proof construction; rich marble work; eight elevators.	42
Ernest and Cramer Building, Denver.	1890	Eight stories; flat roof; brick front; mill construction; oak finish; three elevators.	19
Bailey Block, Denver.	1890	Three stories; flat roof; one front store facing; ordinary brick and timber construction; plumbing and steam heat; pine finish.	8½
Crocker Building, San Francisco.	1890	Ten stories; flat roof; brick and terra-cotta fronts; skeleton construction; fire-proof; elaborate finish, marble, etc.	63
Bradbury Building, Los Angeles, Cal.	1891	Five stories; flat roof; buff brick and terra-cotta walls; fire-proof construction; oak finish; two elevators.	32
Endicott Building, St. Paul, Minn.	1887–9	Seven stories; flat roof; pressed-brick front; fire-proof construction; marble wainscot; five elevators.	29
— Office Building, Connecticut (R. W. Gibson).	1891	Three stories; two stone fronts; fire-proof; usual plumbing, heating plant, fixtures, etc.; rich marble work; stories of moderate height	50
Five Office Buildings in Minnesota.	1893–6	Eight- to twelve-story buildings about of the character of the Rookery, Chicago.	29½–35
Board of Trade Building, Duluth, Minn.	1895	Seven stories; two fronts; fire-proof; handsomely finished; equal to the Old Colony in Chicago.	38
Seven-story Office Building, Montreal.	1903	One front; fire-proof; about the type of the Union Trust Building in St. Louis.	36
Ten-story Office Building, Chicago.	1903	Equal to Old Colony Building.	37½

* Jenney and Mundie, architects.

ACTUAL COST OF BUILDINGS—(*Continued*).

Warehouses and Stores.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
Eight-story Office and Bank Building, San Francisco	1902	One front; fire-proof; equal to the Brown Hotel, Denver.	39
Eight-story Bank and Office Building, Atlanta.	1904	Fire-proof; quite elaborate.	41
Warehouse, Minnesota.	1896	Five stories, fire-proof; for very heavy goods; good front; steam heat; plenty of elevators, etc.	17⅛
Seven-story Warehouse, Minnesota.	1898	Mill construction; plain brick walls.	9 1/13
Seven-story Warehouse, Cincinnati.	1904	Fire-proof; cement floors; no finish.	25⅙
Store Building, New Orleans.	1903	Four stories; fire-proof; plain finish.	31
Department Store, Chicago.	1900	Six stories; fire-proof construction; one front, modern.	29
Leiter Building, Chicago.*	1892	Wholesale and retail store; eight stories; granite three sides; brick on alley.	19¾

Hotels and Apartment Buildings.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
— Hotel, New York (R. W. Gibson).		Fourteen stories; brick and terra-cotta front; skeleton construction, riveted; fire-proof; usual plumbing, machinery, etc.	44
Brown Palace Hotel, Denver.	1892	Triangular plan; three stone fronts; considerable carving; nine stories; flat roof; all rooms face street; 350 guest rooms, 160 private baths, 17 public toilet rooms, all tiled; steel construction; fire-proof; provided with electric light, ice and refrigerator plant; laundry; 4 elevators.	30
Eight-story Apartment House, New York.	1901	Fireproof; elaborately finished.	42
Two Apartment Houses, New York	1903	Fire-proof, but no more elaborate than above.	51
Seven-story Apartment House, Pittsburg.	1903	Fire-proof; hard-wood finish; not elegant; two elevators.	39⅜
The Lenox (Apartments), Cleveland, Ohio.	about 1889	Five stories; flat roof; pressed-brick front; partly slow-burning construction.	18½

Club Buildings, Y. M. C. A., Etc.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
Athletic Club Building, Denver, Colo.	1890–1	Four stories; flat roof; one front pressed brick; thoroughly equipped with swimming and Turkish baths, gymnasium, hand-ball room, billiard-room, social rooms, etc.; brick walls, wood construction.	18

* Jenney and Mundie, architects.

ACTUAL COST OF BUILDINGS—(*Continued*)

Club Buildings, Y. M. C. A., Etc.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
Denver Club Building, Denver, Colo.	1887–8	Three stories and high-pitch roof; stone ashlar, four sides; slate roof; wood construction; oak and pine finish.	24
Standard Club Ho., Michigan Avenue, Chicago.	1887	....	12 9/10
Y. M. C. A. Building, Cleveland, O.		....	13

School, College, and Seminary Buildings.

Wingate Hall, State College, Orono, Me.	1891–2	Three stories and basement; recitation- and drawing-rooms; brick with granite trimmings; slate roof; wood floors; brick partitions.	10½
Grammar-school Building, Denver, Colo., 8 rooms.	1891–2	Two stories and basement; pressed-brick walls; shingle and tin roof; wooden floors; brick partitions; cost, basement floor to second-story ceiling.	9½
Smedley School, Denver, 4 rooms.	1902	Pressed brick; wooden-floor construction; shingle roof; slate blackboard; janitor's rooms in basement; two large furnaces.	10⅘
Clayton School, Denver, 15 rooms, 2 lunch-rooms, 6 rooms for janitor, and large hall in attic.	1901	Light pressed brick; wooden-floor construction; otherwise first-class building; fan system heating and ventilation.	10
Ursuline Convent, Cleveland, O.	1890	Three stories; pitch roof; brick with stone trimmings; ordinary wood construction.	15
Hill Theological Seminary, St. Paul, Minn.		Six buildings grouped around a quadrangle; ordinary construction; library, gymnasium, and staircases fire-proof; corridor walls face brick; oak finish; cost per cubic foot *above grade*.	11
Leland Stanford Jr. Museum, Palo Alto, Cal.	1891	This building, covering 21,000 feet and containing over 1,100,000 cubic feet of space, is built entirely of Portland-cement concrete—walls, floors, and roof—and is fireproof throughout.	18
Newark High School, Newark, N. J.	1897–8	A large three-story building, mostly fire-proof construction. Cubical contents from basement floor to mean point in roof, 1,803,000 cu. ft. For description of building, list of contracts, etc., see *American Architect*, July 9, 1898.	10¾

ACTUAL COST OF BUILDINGS—(*Continued*).

School-houses.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Room.	Cost per Cu. Ft. Cts.
St. Louis.				
Eugene Field School		All first-class buildings, described in *Brickbuilder* for October, 1903.	$5,600	15¾
Edward Wyman Sch.			5,600	14
Horace Mann School.			6,007	14$\frac{9}{10}$
Ralph Waldo Emerson School.			5,636	14¾
Cote Brilliante School			6,758	17
Henry Blow School.			6,243	16

School-houses of entirely fire-proof construction, built in Boston, 1892–1903, cost from 22.39 cts. per cu. ft. for the South Boston High School to 24.98 cts. for the Heath Street School. The Dorchester High School, which is fire-proof construction except for a plank roof, cost 16.33 cts. Schools of ordinary construction range from 16.58 to 24 cts. per cu. ft.—*Brickbuilder*, August, 1903.

Libraries.

Public Library, New London, Conn.	1889–90	One-story stone building; ordinary construction.	36½
Howard Memorial Library, New Orleans, La.	1888		44
Congressional, Washington, D.C.		Including some of its furnishings.	67¼

Hospitals.

Hospital Building, New York (R. W. Gibson).	1890–5	Seven stories; pressed-brick front; stone trimmings; fireproof; thorough heating and ventilating plant; plumbing; much marble and tiling.	40
Hospital Building, New York (R. W. Gibson).	1890–5	Six stories; pressed-brick front; stone trimmings; part fireproof and part non-fireproof, but with metal lathing and terra-cotta furring; plumbing, steam plant, etc.	32

Churches.

Grace M.E. Church, Cambridgeport, Mass.	1886–7	Two-story wooden building; tower and spire; slate roof; copper metal-work; cost includes furnaces, pews, frescoing, and gas-fixtures.	8¾
Christ M.E. Church, Denver, Colo.	1889–91	Two-story stone church; stone tower 71 feet high, with wood spire 108 feet high, above; shingle roof; steam heat; oak finish in second story; pews, frescoing, etc.	21
Zion Temple, Synagogue, Ogden Av., Chicago.	1885	....	7$\frac{9}{10}$

ACTUAL COST OF BUILDINGS.—(*Continued*).

Theatres.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
Theatre, Duluth, Minn.*	1893	Six stories; brick, stone, and terra-cotta; two fronts absolutely fireproof and elegantly decorated and finished.	33⅙
Schiller Building, or German Theatre, Chicago.	1891	Seventeen stories; flat roof, faced with terra-cotta; skeleton construction; fireproof; rich marble work; theatre in four stories.	30 8/10

Miscellaneous.

Name of Building.	Date.	Character of Construction and Finish.	Cost per Cu. Ft. Cts.
Park pavilion.......	1898	Built in middle West; all wood and glass; two stories, dining-room, dancing-hall, etc.	4 7/10
Large car barn......	1895	Exposed iron construction and brick walls.	9 1/13
American Express stables, Chicago. †	1893	Steel construction, fire-proofed, Sioux Falls, Jasper; first story, pressed brick above; tile arches; four stories and basement.	10⅕

* Traphagen & Fitzpatrick, architects. † Jenney & Mundie, architects.

Dwellings.

(See also pages 1458, 1460.)

City dwellings in Chicago, designed by Adler & Sullivan, architects. Cost per cubic foot from..17 to 20 cts.

Of dwellings designed by the author and built in Boston in 1886, the average cost of eight- and ten-room wooden houses per cubic foot of habitable space, including cellar, was about....... 11 cts.

In Denver, Colo., the cost of a first-class stone house (isolated), with hard-wood finish, indirect steam heat, extra plumbing, decorations, etc., complete, was in 1890 about.................. 27 cts.

Brick houses of ten rooms, pine finish, furnace heat, good plumbing, etc., cost above cellar floor, but not including unoccupied roof space, in 1892.. 14 cts.

Cheap eight-room brick cottages of one and one-half or two stories; bath-room and furnace; cubic space reckoned from cellar floor, but not including unoccupied roof space, were built in Denver, in 1894, for about.. 10 cts.

Cost of Different Kinds of Work per Cubic Foot of Building.

In *Fireproof* for March, 1903, Mr. F. W. Fitzpatrick gave some figures showing the proportionate cost of the different branches of work which go to make up the completed building. Believing that these data will be found useful in making up approximate estimates, the author obtained permission to use them herein.

The following figures represent the actual cost of a prominent *ten-story office building*, 60′×130′, built in the middle West, a No. 1 high grade fire-proof structure, with two street fronts faced with granite; pile foundation.

(Figures are in cents per cubic foot of building.)

The foundation cost	$1\frac{3}{4}$
Steel framing	$2\frac{1}{2}$
Granite and all masonry	$11\frac{1}{6}$
Cornice, roofs, and skylights	$\frac{2}{3}$
Fire-proof floors	$\frac{2}{3}$
Partitions (tile)	$\frac{2}{5}$
All plastering and stucco	$1\frac{1}{4}$
Elevator fronts and all ornamental metal-work	2
Marble-work	$3\frac{1}{6}$
Hardware	$\frac{2}{15}$
Joiner work	$1\frac{1}{6}$
Glass	$\frac{5}{12}$
Painting and varnish	$\frac{7}{30}$
Electric wiring	$\frac{2}{3}$
Heating	$1\frac{1}{8}$
Plumbing	$\frac{1}{2}$
Elevators	1
Stairs, scenic structural framing, "making ends meet," lamp fixtures, etc. What might be called a fair amount for "contingencies" in such a building, including lesser items not mentioned here but grouped together	$4\frac{23}{120}$
Architect's fee	$1\frac{3}{5}$
Total	$34\frac{5}{12}$

The Chicago post-office, a building of 12,000,000 cubic feet and of monumental character and finish, cost, in some of its items, as follows:

(Figures are in cents per cubic foot of entire building.)

Its foundation cost	$1\frac{3}{4}$
The steel framing	$2\frac{1}{2}$
Granite and masonry	$13\frac{1}{2}$
Fire-proof floors	$\frac{2}{3}$
Plaster, plain and ornamental	$1\frac{7}{8}$
Ornamental metal-work	$2\frac{1}{8}$
Marble	$5\frac{2}{3}$
Plumbing	$\frac{1}{2}$
Heating	$1\frac{1}{8}$

It may be noticed that the relative cost of several of these items was identically the same as in the office building. The total cost of this building was $42\frac{1}{8}$ cts. per cubic foot.

COST OF BUILDINGS PER SQUARE FOOT.

One-story buildings of large area, such as exposition buildings, etc., may be estimated almost as accurately by the square foot as by the cubic foot, as there are few or no interior partitions, and usually no plastering or interior finish.

Iron Buildings.—"Roughly speaking, the cost of one-story iron buildings, complete, is, for sheds and storage-houses, 40 to 60 cts. per square foot of ground, and for such buildings as machine-shops, foundries, and electric-light plants, that are provided with travelling cranes, the cost is from 60 to 90 cts. per square foot of ground covered." (H. G. Tyrrell.)

Textile Factories.—See pages 723–725.

Exposition Buildings.—The cost of the *World's Fair buildings* (Chicago, 1893) per square foot of ground covered. including sculpture and decoration, as given by E. C. Shankland, chief engineer, was as follows:

Manufactures and Liberal Arts Building	$1.39
Transportation Building	1.08
Electricity Building	1.69
Machinery Hall	2.12
Agricultural Building	1.44
Administration Building	9.18
Horticultraul Building	1.41
Mines and Mining Building	1.04
Fisheries Building	2.35
Forestry Building	.75

Cost of Structures for the St. Louis Exposition (1904).—The following figures are issued by Isaac S. Taylor, Director of Works, of the World's Fair, showing the area and cost of the principal exhibition buildings. The total area of twenty-two buildings is 123.51 acres, and the total cost $6,939,-992.26. The cost is for the bare buildings, and does not include sculptural or other decorations, or the architect's compensation.

	Dimensions.	Area in Acres	Cost.	Cost per Sq. Ft.
Art Building	161′ × 346′	1.42	$967,833.90	$5.45
Two Art Pavilions, each	144′ × 423′	3.14		
Art Building Annex	106′ × 150′	0.41	39,388.99	2.48
Government Building	200′ × 736′	3.86	328,980.00	2.23
Government Fisheries Building	136′ × 136′	0.42	45,000.00	2.43
Mines and Metallurgy	525′ × 750′	9.08	488,848.50	1.24
Liberal Arts	525′ × 750′	8.80	471,820.95	1.20
Education and Social Economy	525′ × 758′	7.70	323,950.75	0.81
Manufactures	525′ × 1,200′	13.47	711,510.00	1.13
Electricity	525′ × 758′	6.67	408,531.57	1.03
Varied Industries	525′ × 1,200′	10.28	704,067.96	1.12
Machinery	525′ × 1,000′	9.48	509,110.50	0.97
Steam, Gas, and Fuel Building	301′ × 326¾′	2.25	135,480.00	1.38
Transportation	525′ × 1,300′	15.70	674,853.42	0.99
Horticulture	374′ × 782′	5.42	225,342.27	0.77
Agriculture	500′ × 1,600′	18.62	520,491.07	0.58
Forestry, Fish, and Game	300′ × 600′	4.07	168,883.38	0.94
Festival Hall	195′ in diameter, exclusive of annex.	1.09	215,899.00	

Cost of United States Government Buildings.—There was published in 1900, by the Treasury Department, a history of the public buildings, giving the cost, and in the *Architects and Builders' Magazine* for Aug. 1902 and the *Inland Architect* for April 1902, was published a list of 287 buildings, giving the cost per cubic foot, material used for walls, and date of erection. As a rule these buildings have cost more than private buildings, so that their cost cannot be used as a guide, except for government buildings.

DEPRECIATION OF BUILDINGS.

TIFFANY'S ESTIMATE OF DEPRECIATION. (Used by U. S. Gov't.)

The figures given on page 1458 are for NEW buildings. To ascertain the present value, a discount between old and new should be made as follows:

	Per Cent per Year.
Brick, occupied by owner	1 to 1¼
Brick, " " tenant	1¼ to 1½
Frame, " " owner	2 to 2½
Frame, " " tenant	2½ to 3

If built of "long-leaf" yellow pine, or of spruce, found in New England States, add 20 to 30 per cent., or if of "short-leaf" yellow pine, add 40 to 50 per cent. to his figure. If, of redwood or cedar, found on Pacific Coast, charge only about half his estimates, which are for white pine or *white pine* with oak framing timbers.

These figures for depreciation are to include buildings where ordinary repairs have been made. If extraordinary repairs have been made, the discount should not be so heavy. Exercise good judgment as to depreciation.

The Wear and Tear of Building Materials.—At the tenth annual meeting of the Fire Underwriters' Association of the Northwest, held at Chicago in September, 1879, Mr. A. W. Spalding read a paper on the wear and tear of building materials and tabulated the result of his investigations in the following form:

Material in Building.	Frame Dwelling.		Brick Dwelling (Shingle Roof).		Frame Store.		Brick Store (Shingle Roof).	
	Average Life, Years.	Per Cent. of Depreciation per Annum.	Average Life, Years.	Per Cent. of Depreciation per Annum.	Average Life, Years.	Per Cent. of Depreciation per Annum.	Average Life, Years.	Per Cent. of Depreciation per Annum.
Brick	.....	.....	75	1⅛	.....	.....	66	1½
Plastering	20	5	30	3⅓	16	6	30	3⅓
Painting, outside	5	20	7	14	5	20	6	16
Painting, inside	7	14	7	14	5	20	6	16
Shingles	16	6	16	6	16	6	16	6
Cornice	40	2½	40	2½	30	3⅓	40	2½
Weather-boarding	30	3⅓	.....	.....	30	3⅓		
Sheathing	50	2	50	2	40	2½	50	2
Flooring	20	5	20	5	13	8	13	8
Doors, complete	30	3⅓	30	3⅓	25	4	30	3⅓
Windows, complete	30	3⅓	30	3⅓	25	4	30	3⅓
Stairs and newel	30	3⅓	30	3⅓	20	5	20	5
Base	40	2½	40	2½	30	3⅓	30	3⅓
Inside blinds	30	3⅓	30	3⅓	30	3⅓	30	3⅓
Building hardware	20	5	20	5	13	8	13	8
Piazzas and porches	20	5	20	5	20	5	20	5
Outside blinds	16	6	16	6	16	6	16	6
Sills and first-floor joists	25	4	40	2½	25	4	30	3⅓
Dimension lumber	50	2	75	1⅓	40	2½	66	1½

These figures represent the averages deduced from the replies made by eighty-three competent builders unconnected with fire-insurance companies in twenty-seven cities and towns of the eleven Western States.

DIMENSIONS AND DATA USEFUL IN THE PREPARATION OF PLANS.

Dimensions for Furniture.—For the convenience of draughtsmen when designing furniture or providing space for a special article the following dimensions are given: *

Chairs and Seats.—The average figures taken from a variety of good chairs are: Height of the seat above the floor, 18″; depth of the seat, 19″; the top of the back above the floor, 38″. Usually the seat increases in depth as it decreases in height, while the back is higher and slopes more. Twenty inches inside is a comfortable depth for a seat of moderate size. Chair-arms are about 9″ above the seat. The slope of the back should not be more than one fifth the depth of the seat. A *lounge* is 6′ long and about 30″ wide.

Tables vary in shape and size almost as much as chairs. Writing- and dining-tables are made 2′ 5″ high, and the species of sideboard called a carving-table is made 3′ high to the principal shelf but tables for general use are 2′ 6″ high.

Dining-tables are made from 3′ 6″ to 4′ wide and to extend from 12′ to 16′ feet by means of slides within the frame. This frame should not be so deep as to interfere with the knees of any one sitting at the table; that is, there must be about 2′ clear space between it and the floor.

The smallest size practicable for the knee-holes of desks and library tables is 2′ high by 1′ 8″ wide, the width to be increased as much as possible.

Bedsteads are classed as single, three quarters, and double. A single bed is 3′ to 4′ wide inside; a three-quarter bed, 4′ to 4′ 6″; a double bed, 5′. All bedsteads are 6′ 6″ to 6′ 8″ long inside. Footboards are from 2′ 6″ to 3′ 6″ and headboards from 5′ to 6′ 6″ high. Single beds for dormitories are often made only 2′ 8″ wide.

Bureaus vary in shape and size to such an extent that it is impossible to say that any dimension is fixed.

Convenient sizes are: body, 3′ 5″ wide, 1′ 6″ deep, 2′ 6″ high; or 4′ wide, 1′ 8″ deep, 3′ high.

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* Many of these dimensions were first contributed to the *American Architect* of November 10, 1894, by Mr. Alvin C. Nye.

Commodes are 1′ 6″ square on the top and 2′ 6″ high.

Chiffoniers are about 3′ wide, 1′ 8″ deep, 4′ 4″ high.

Cheval glasses are made, if large, 6′ 4″ high, 3′ 2″ wide. If small, 5′ high, 1′ 8″ wide. If medium, 5′ 6″ high, 2′ wide.

Wash-stands of large sizes are 3′ long, 1′ 6″ wide, and 2′ 7″ high. Small sizes are 2′ 4″ to 2′ 8″ long.

Wardrobes may be 8′ high, 2′ deep, and 4′ 6″ wide; or, 6′ 9″ high, 1′ 5″ deep, and 3′ wide.

Sideboards may be 4′ to 6′ long and from 20″ to 2′ 2″ deep.

Upright pianos vary from 4′ 10″ to 5′ 6″ in length, from 4 to 4′ 9″ in height, and are about 2′ 4″ deep over all.

Square pianos are about 6′ 8″ long by 3′ 4″ deep.

Billiard-tables (Collender), 4′×8′, 4′ 2″×9′, and 5′×10′. Size of room required 13′×17′, 14′×18′, and 15′×20′ respectively.

Dimensions of Plumbing Fixtures.—*Enamelled-iron Bath-tubs.*—Standard sizes for roll-rim baths with sloping end are: Nominal lengths, 4′, 4½′, 5′, 5½′, and 6′; width over all, 30″ to 34″. Specially narrow tubs are made 25″ to 29″ wide. The actual length over rim is usually 1″ or 2″ more than the nominal length, and 2″ will include ordinary overflow-pipe.

Wash-basins.—Crockery basins, to go with marble slabs, are made round and oval. Round bowls are made 10″, 12″, 13″, 14″, and 16″ in diameter, measured from the outside of the rim. Oval bowls, 14″×17″, 15″×19″, and 16″×21″. The 12″ and 14″ round, and 15″×19″ oval, are most commonly used.

Marble basin-slabs may be 20″×24″, 20″×30″, 22″×28″, or 24″×30″, the last being a very common size. Can be made any size to order. They should be 1¼″ thick, countersunk on top, and should have moulded edges where exposed.

Corner slabs are commonly made 21″×21″ and 24″×24″. Marble backs are usually 8″ or 10″ high, and sometimes 12″.

Enamelled-iron wash-basins or *lavatories* made in one piece: Common sizes are 16″×20″, 11″×14″ basin; 18″×21″, 11″×15″ basin; 18″×24″, 12″×15″ basin; back, 10½″ high. The smallest-size wash-basin is 13″ wide at the back.

Corner basins, 12½″×12½″, 12″ round basin; 15″×15″, 11″×14″ basin, 16″×16″, 11″×14″ basin, 19″×19″, 11″×15″ basin. The standard height of wash-basins is 2′ 6″ from the floor.

Foot-baths, enamelled iron, roll rim, are 22½″×19″; width, including fittings, 1′ 11″; height 17″; depth inside 11″.

Seat-baths, enamelled iron, average about 32″ long over fittings, and 27″ wide.

Water-closets.—The dimensions of water-closet bowls vary considerably, the following being about an average: Width of bowl over all, 13″; depth from wall to front of seat, 23″; height from floor to seat, 17″; width of seat, 15″ to 16″. Closets with low-down tanks measure about 28″ from front of seat to wall. The distance from centre of outlet opening to the walls, or the "roughing in" dimensions, are given in manufacturers' catalogues, as they vary with different closets. The smallest space permissible for water-closet compartments, where doors open out, is 2′ 4″×4′ 0″. If the door opens in, the compartment should be 3′×5′.

Closet-ranges, used in schools and factories, are made 24″, 27″, and 30″, centre to centre of partitions. For grade schools, 24″ is ample, and for factories, 27″. The range usually occupies a space 28″ in depth if set against a wall.

Urinal-stalls should be 24″ to 27″, centre to centre of partitions, depth of partitions, 20″ or 22″; of ends, 2′; of bottom slab, 2′; height of partitions, 4′ 6″ to 5′ 6″.

Kitchen-sinks of cast iron are made in a great variety of sizes, those most commonly used being 16″×24″, 18″×30″, 18″×36″, 20″×30″ and 20″×36″; 24″×50″ being the largest size for enamelled sinks. The depth inside for the sizes given is 6″. Plain cast-iron sinks are made as large as 32″×56″ or 28″×78″. Steel sinks are made in all of the above sizes up to 20″×40″.

Common sizes of *Porcelain sinks* are 20″×30″, 23″×36″, 24″×42″.

Cast-iron slop-sinks, common sizes, are 16″×16″, 16″×20″, 18″×22″, 20″×24″; 12″ deep.

Copper Pantry-sinks.—Common sizes are 12″×18″, 14″×20″, and 16″×24″.

Laundry-tubs of slate or soapstone are commonly made 2′ wide over all, and 16″ in depth. Lengths over all, two-part tubs, 4′ 0″ and 4′ 6″; three-part tubs, 6′ 0″, 6′ 6″, and 7′.

Earthen and porcelain tubs come separately, and are connected up as required.

The dimensions of each tub are 2′ or 2′ 7½″ in length, 2′ 1½″ in width, and 15″ in depth inside.

The length required for two 2′ tubs is 4′ 1″; for three tubs, 6′ 2″; and for four tubs, 8′ 3″.

Wolff's roll-rim enamelled-iron wash-tubs are 55″ over all, for two tubs, and 82″ for three tubs.

Range-boilers are 12″ diameter for 30-gallon boilers, 14″ for 40-gallon, 16″ for 52- and 63-gallon, 22″ for 100- and 120-gallon.

Dimensions of Carriages.—*Covered Buggy (Goddard).*—Length over all, 14′; width, 5′; height, 7′ 4″. Will turn in space from 14′ to 20′ square, according to skill.

Coupé.—Length over all, 18′; width, 6′; height, 6′ 6″.

Buggy (Piano Box).—Length over all, 14′; width, 4′ 10″.

Landau.—Length over all, 19′ 6″; width, 6′ 3″; height, 6′ 3″; length of pole, 8′ 0″.

Stanhope Gig (2 Wheels).—Length over all, 10′ 6″; width, 5′ 8″; height, 7′ 6″.

Victoria.—Length, without pole, 9′ 6″; length of pole, 8′; width over all, 5′ 4″.

Light Brougham.—Length, without pole or shaft, 9′ to 11′; width over all, 5′ 4″; height, 6′ 4″.

Dimensions and Weight of Fire-Engines.—From measurements of different fire-engines belonging to the city of Boston, it was found that the greatest length, including pole, was 22′ 6″. The widths varied from 5′ to 5′ 11″, the average height being 8′ 8″.

The average weight of 29 engines is 8,000 lbs.; the greatest weight being 9,420 lbs., and the least 4,780 lbs.

Dimensions and Weight of Hose Carriages.—Extreme length with horse, 19′ 6″, without horse, 17′ 6″; width, 5′ 9″ to 7′ 0″; height, from 6′ 8″ to 7′ 0″; average weight of 11 carriages, 2,943 lbs.; greatest weight, 3,500; least weight, 2,120.

Dimensions and Weight of Ladder Wagons.—Length of truck, 33′; total length, with ladders on, 45′; width, 6′ 2″; average weight of 12 wagons, 6,660 lbs.; greatest weight, 8,800; least, 4,350.

Dimensions of Locomotives and Cars.—The dimensions of locomotives and freight-cars vary considerably, but the following will cover those in common use:

Locomotives.—15′ 4″ to 15′ 10″ to top of stack from top of rail; extreme width of cab, 10′ 2″. Doors to admit locomotives should be 12′ to 13′ wide and 18′ high.

Furniture-cars are 14′ 1″, top of track to top of brake-staff; floor 3′ 8″ from track; extreme width, 9′ 10″.

Stock-cars, 13′ 5″, top of track to top of brake-staff; floor, 4′ 0″ from track; extreme width, 9′ 8″.

Refrigerator-cars, 14′ 6″, top of track to top of brake-staff; floor, 4′ 0″ from track; extreme width, 9′ 7″.

Ordinary freight-cars are about 13′ 0″ high to top of brake-staff and 9′ 4″ in extreme width. The height of floor of freight-cars varies from 3′ 8″ to 4′ 0″ above top of track, for standard gauge, and 3′ 0″ to 3′ 6″ for narrow-gauge cars.

Passenger-coaches vary from 14′ to 16′ in height and 10′ to 11′ in width. Doors to admit cars should give at least 12″ clearance on each side, and 2′ overhead.

Street trolley-cars are about 8′ 6″ wide for the car proper, and the steps project about 8″. Height from track to top of coach, 11′ 6″; trolley-stand is 18″ higher. Length up to 42′. Trucks for a 41′ 6″ car are about 24′ apart. Wheel-base, 4′ centre to centre. Radius of shortest curve in Denver, 35′ to midway between rails.

The **gauge** of a railroad track is the distance between the inner sides of the heads of the two rails. The standard or "broad" gauge is 4′ 8½″; standard narrow gauge, 3′ 3½″.

Capacity of Freight-cars.—*Car-loads.*—The capacity of freight-cars, and the minimum car-load, varies so greatly that no accurate general information can be given. For heavy freight, 25 tons is an average load; for light freight, 12 to 15 tons; for household goods, 10 tons is about the minimum; for lime, 15 tons is about a minimum load; for cement, 20 tons. The minimum car-load, to obtain car-load rates, varies with different roads, and also with the rate made; a low rate is usually made on the basis of a big load. Thirty tons is a good load for heavy freight, and 40 tons is about the maximum, except for special cars.

Miscellaneous Dimensions.—*Horse-stalls.*—Width, 3′ 10″ to 4′, or else 5′ or over in width, 9′ long. Width should never be between 4′ and 5′, as the horse is liable to cast himself.

Dimensions of Standard Bowling Alley.—Length of alley, 60′; width from centre to centre, 5′ 6″; width of bed, 3′ 6″; troughs each side. Pit 5′; to drop one foot at end below level of bed; runway, 11′ 6″.

Dimensions of Drawings for Patents (United States).—10″×15″ with border-line 1″ inside all around.

Dimensions of a Barrel.—Diameter of head, 17″; bung, 19″; length, 28″; volume, 7,680 cu. ins.

Miscellaneous Memoranda.—*Weight of Men and Women.*—The average weight of twenty thousand men and women weighed at Boston, 1864, was,—men, 141½ lbs.; women, 124½ lbs.

Avenues of City of New York run 28° 50′ 30″ east of north.

Flagpoles.—For a flagpole, extending from 30′ to 60′ above the roof, the following proportions give satisfactory results: The diameter at the roof should be $\frac{1}{50}$ the height above the roof, and the top diameter $\frac{1}{2}$ the lower. To profile the pole, divide the height into quarters; make the diameter at the first quarter above the roof, $\frac{15}{16}$ of the lower diameter; at the second quarter, $\frac{7}{8}$, and at the third quarter, $\frac{3}{4}$ the lower diameter.*

Dimensions of Schoolrooms, Boston Schools.—The sizes of the rooms in the Boston schools, as adopted by the School board, are, for grammar schools, 28′×32′×13′ 6″ high; for primary schools, 24′×32′×12′. This accommodates 56 scholars per room, in each grade, allowing 216 cu. ft. per scholar in the grammar schools, and 165 cu. ft. in the primary grade.

A width of 27′ is very satisfactory for schoolrooms, and is commonly adopted because it permits of the use of 28′ joists without waste.

Height of Blackboards in Schoolroom.—The height from floor to top of chalk-rail should be about as follows:

3d and 4th grades,	chalk-moulding..........	2′ 1″	from floor
5th grade,	"	2′ 2½″	" "
6th grade,	"	2′ 4″	" "
7th and 8th grades,	"	2′ 6″	" "

Slate blackboards are made 3′ 6″, 4′ 0″, and 4′ 6″ high, 4′ being a very common and satisfactory height.

SIZES OF CHAIRS AND DESKS FOR SCHOOLS AND ACADEMIES.

Age of Scholar.	Height of Chair.	Height of Desk (Next Scholar).	Space Occupied by Desk and Chair (Back to Back of Desk).
16 to 18 years.	16¾ inches.	29½ inches.	2 feet 9 inches.
14 to 16 "	15½ "	28 "	2 " 9 "
12 to 14 "	15½ "	27½ "	2 " 8 "
10 to 12 "	14½ "	26½ "	2 " 7 "
8 to 10 "	13½ "	25½ "	2 " 5 "
7 to 8 "	12½ "	24 "	2 " 4 "
6 to 7 "	11½ "	22½ "	2 " 3 "
5 to 6 "	10½ "	21 "	2 " 2 "
4 to 5 "	9½ "	19 "	2 " 0 "

Desks for two scholars are 3 ft. 10 ins. long, and for a single scholar, 2 ft. long.

Aisles are 2 ft. to 2 ft. 4 ins. wide, according to age of scholars and size of room.

* The Building Trades Pocket-book.

Stairs.*—The *"rise"* of a stair is the height from the top of one step to the top of the next. The "total rise" is the height from floor to floor. The *"run"* is the horizontal distance from the face of one riser to the face of the next. *"Risers"* are the upright boards forming the face of the steps, and the *"treads"* are the horizontal boards on which the feet tread. Treads are usually from 1¼ to 1¾ ins. wider than the run, on account of the nosing.

The "rise" of any stairs is found by dividing the "total rise" by the number of risers. The "run" of the stairs may be fixed at will unless the space is cramped, but to secure a comfortable stair the run must bear a certain relation to the rise.

Rules for Proportion of Treads and Risers.—For ordinary use a rise of 7 to 7½ ins. makes a very comfortable stair. In schools and for stairs used by children the rise should not exceed 6 ins. Stairs having a rise greater than 7¾ ins. are steep.

The width of the run should be determined by the height of the rise; the less the rise the greater should be the run, and *vice versa.* Several rules have been given for proportioning the run to the rise, viz.:

(1) *The sum of the rise and run* should be equal to from 17 to 17½ ins.

(2) *The sum of two risers and a tread* should not be less than 24 nor more than 25 ins.

(3) *The product of the rise and run* shall not be less than 70 nor more than 75.

These rules apply only to stairs with nosings. Stone stairs without nosings should have at least 12-in. treads to be comfortable for adults.

Height of Hand-rail.—In dwellings, hotels, apartments, etc., the height of the rail should be about 2 ft. 6 ins. above the tread, on a line with the face of the riser. For grand staircases the height may be reduced to 2 ft. 4 ins. On steep stairs the height should be from 2 ft. 7 ins. to 2 ft. 9 ins. The rail should also be raised over winders. On landings the height of the rail should be equal to the height of the stair-rail measured at the centre of the tread, the usual height in residences being 2 ft. 8 ins. to 2 ft. 10 ins.

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* This subject is quite fully treated in Part II, Building Construction and Superintendence.

Sash-weights.—The weights ordinarily used for balancing windows are made of a cheap grade of cast iron, in the form of a solid cylinder, with an eye cast in the upper end. On account of the limited space in the weight-box, the weights should never be of greater diameter than the thickness of the sash; consequently for balancing heavy sash, the weights must be quite long. For wide and low windows the ordinary sash-weight may be too long to permit the sash to be raised or lowered its entire height, and in special cases it is often necessary to use square sash-weights, and frequently it is necessary to use lead weights, lead being about 58 per cent. heavier than cast iron.

The length of sash-weight in inches required to balance a given weight may readily be found by dividing the given weight by the values in the following table. To the quotient thus obtained 1 in. should be added to cover the eye.

WEIGHT OF IRON AND LEAD SASH-WEIGHTS PER LINEAL INCH.

Diameter, or Side of Square.	Round Cast Iron.	Square Cast Iron.	Square Lead.
Inches.	Pounds.	Pounds.	Pounds.
$1\frac{1}{4}$	0.32	0.40	0.64
$1\frac{1}{2}$	0.46	0.58	0.92
$1\frac{3}{4}$	0.62	0.79	1.25
2	0.81	1.04	1.64
$2\frac{1}{4}$	1.03	1.31	2.07
$2\frac{1}{2}$	1.27	1.62	2.56
$2\frac{3}{4}$	1.54	1.96	3.10
3	1.83	2.34	3.69
$3\frac{1}{2}$	2.50	3.18	5.02
4	3.26	4.16	6.56

The approximate diameters of the common stock cast-iron sash-weights are: for weights of 8 lbs. and under, $1\frac{1}{2}$ ins.; for weights between 8 and 16 lbs. inclusive, $1\frac{3}{4}$ ins.; for weights between 16 and 20 lbs.; 2 ins. and for weights between 20 and 30 lbs., $2\frac{1}{4}$ ins. Weights above 30 lbs. are usually square in cross-section.

Weight of Sash and Glass.—For approximating the weight of windows, the weight of the glass may be taken at $3\frac{1}{2}$ lbs. per square foot for plate glass, $1\frac{1}{3}$ lbs. for double-strength glass, and 1 lb. for single-strength glass.

For the weight of the wooden sash, add together the height

and width of each sash (in feet) and multiply by 2.1 for $2\frac{1}{4}$-in. sash, $1\frac{3}{4}$ for $1\frac{3}{4}$-in. sash, and $1\frac{1}{3}$ for $1\frac{3}{8}$-in. sash.

The exact weight, however, can only be obtained by weighing each sash, as the glass varies considerably in weight.

In hanging sashes, the weights for the upper sash should be about $\frac{1}{2}$ lb. heavier than the sash, and for the lower sash $\frac{1}{2}$ lb. lighter.

Seating Space in Churches and Theatres.—The minimum spacing for pews, back to back, is 30 ins. This spacing is fairly comfortable to occupants, but is a little cramped for persons to pass by others into or out of the pew. A spacing of 32 ins. is to be preferred, and if there is abundance of room, the spacing may be made 33 ins.; anything over 33 ins. is a waste of room. 18 ins. in the length of the pew is considered as a "sitting."

For dimensions of pew bodies see p. 48 of "Churches and Chapels."

Opera or Assembly Chairs are made 19, 20, 21, and 22 ins. wide, centre to centre of arms, and in arranging them in rows where the aisles converge, the ends are brought to a line on the aisles by using a few chairs that are either narrower or wider than the standard width. For churches, a standard width of 20 ins. is the least that is desirable.

For theatres, 21- or 22-in. chairs are commonly used in the parquet, 20- or 21-in. in the dress circle, and 20- and 19-in. in *balcony and gallery*, although there is no accepted rule in this respect.

On account of the seat lifting, opera or assembly chairs may be comfortably spaced 30 ins. back to back, and this is the usual spacing in halls and churches.

In theatres the chairs are usually set on steps. In the upper gallery these steps should not be more than 30 ins. wide; in the balcony they are usually made either 30 or 31 ins. wide, and in the parquet 31 or 32 ins. wide. As a rule the higher-priced seats are more commodious than the lower-priced.

Estimating Seating Capacity.—The actual seating capacity of theatres and audience-rooms can be determined only by drawing the seats to an accurate scale, on the floor plan, and then counting the number of chairs, or measuring the lineal feet of pews.

For approximate purposes the seating capacity or required size of room may be determined by allowing from 7 to 8 sq. ft.

to each seat, or sitting, when on a curve, and 6 to 7 sq. ft. to each sitting when in straight rows, the smaller number being used only for large rooms. This allows for aisles and pulpit platform. For small concert halls and narrow rectangular rooms 6 sq. ft. per sitting will usually be sufficient allowance, provided only the actual floor space utilized for seats and aisles is considered.

CAPACITY OF SEVERAL CHURCHES, THEATRES, AND OPERA-HOUSES.

CHURCHES.

(Estimating a person to occupy an area of 19.7 ins. square.)

St. Peter's	54,000	Notre Dame, Paris	21,000
Milan Cathedral	37,000	Pisa Cathedral	13,000
St. Paul's, Rome	32,000	St. Stephen's, Vienna	12,400
St. Paul's, London	25,600	St. Dominic's, Bologna	12,000
St. Petronio's, Bologna	24,400	St. Peter's, Bologna	11,400
Florence Cathedral	24,300	Cathedral of Sienna	11,000
Antwerp Cathedral	24,000	St. Mark's, Venice	7,000
St. Sophia's, Constantinople	23,000	Spurgeon's Tabernacle	7,000
St. John Lateran's	22,900		

THEATRES AND OPERA-HOUSES.

EUROPEAN.

Carlo Felice, Genoa	2,560	Drury Lane, London	1,948
Opera-house, Munich	2,370	Covent Garden, London	3,000
Alexander, St. Petersburg	2,332	Opera House, Berlin	1,636
San Carlos, Naples	2,240	Adelphi, London	2,300
Imperial, St. Petersburg	2,160	Lancaster, London	1,850
La Scala, Milan	2,113	Globe, London	1,100
Academy of Paris	2,092		

AMERICAN.

The Auditorium, Chicago	4,200	Abbey's Theatre, N. Y.	1,450
Metrop't'n Opera House, N. Y.	3,200	Empire Theatre, N. Y.	1,150
Philadelphia Academy	3,124	Fifth Ave. Theatre, N. Y.	1,400
Boston Theatre, Boston	3,000	Castle Square Theatre, Boston	1,600 to 1,800
American Theatre, N. Y.	2,500	Gaiety Theatre, Boston	nearly 3,000
Proctor's Pleasure Palace, N. Y.	2,100	Grand Opera-house, Cincinnati, O.	1,736
Lyric Theater, N. Y.	1,543		

DIMENSIONS OF THEATRES AND OPERA-HOUSES.

The following are the dimensions, in feet, of some of the prominent theatres in this country and in Europe:

Name and Location.	Auditorium.			Proscenium Opening.		Stage.		
	Width.	Depth.	Height.	Width.	Height.	Width.	Depth.[1]	Height.[3]
Alexander, St. Petersburg	58	76	58	56	.....	75	84	
——, Berlin	51	78	47	41	.....	92	76	
La Scala, Milan	71	95	64	49	.....	86	78	
San Carlo, Naples	74	73	83	52	.....	66	74	
Grand Theatre, Bordeaux	47	56	57	37	.....	80	69	
Salle Lepelletier, Paris	66	76	66	43	.....	78	82	
Covent Garden, London	51	66	.....	32	.....	86	55	
Drury Lane, London	56	64	.....	32	.....	48	80	
Boston Theatre, Boston	.....	71	58	46	.....	87	68	
Academy of Music, N. Y.	62	87	.....	48	.....	83	71	
Opera-house, Phila.	66	78	74	48	.....	90	72	
Globe Theatre, Boston	60	65	.....	30	.....	62	38	
Museum, Boston	68	61	.....	31	.....	68	46	
Metrop't'n Opera House, New York[4]	.....	.....	.....	54	50	100	73	88
The Auditorium, Chicago	.....	.....	.....	.....	.....	110	70	95
Empire Theatre, N. Y.	69	66	.....	34	34	67	30	65
Abbey's Theatre, N. Y.	70¾	79	47⅔[2]	35	34	40	65½	
Harrigan's Theatre, N. Y.	56⅙	52	.....	27	.....	71½	28⅓	
Fifth Ave. Theatre, N. Y.	.....	.....	.....	.....	.....	80	35	
American, N. Y.	74½	74½	.....	39	39	77¾	43⅓	73½
Proctor's Pleasure Palace, N. Y.	74⅓	74½	.....	34	34[5]	70	40	70
Hudson Theatre, N. Y.	67⅓	67	.....	32	30	67⅓	30⅔	
Grand Opera-house, Cincinnati	67	69	.....	36	34	67	41	
Castle Square Theatre, Boston[6]	79½	85½	70[2]	40	34	68	45½	
Gaiety Theatre, Boston	77	80⅔	.....	.....	.....	60	42	70

[1] From the curtain or back line of proscenium opening.
[2] Measured from stage to centre of ceiling.
[3] To the "gridiron" or rigging-loft.
[4] As remodelled in 1893.
[5] Can be enlarged to 40′ × 40′.
[6] The plan of this theatre is in the shape of a horseshoe.

Notes on Theatre Dimensions.*—"The utmost distance from the front of the stage to the rear ought not to exceed 75 ft., or the limit the voice is capable of expanding in a lateral direction."

"Measured *from the curtain line*, the Theatre of San Carlos at Naples is 73 ft.; at Bologna 74 ft. Of the London theatres, the

* From "The Planning and Construction of American Theatres," by Wm. H. Birkmire

Adelphi is 74 ft., Covent Garden 80 ft., the Gaiety 53 ft. 6 ins., Lancaster 58 ft. 4 ins., Marylebone 74 ft., and the Globe 47 ft. 6 ins."

The width of the ideal theatre, between inside walls, should be from 70 to 75 ft., and "the ceiling should be 55 to 65 or even 70 ft. above the stage level."

"The depth of the parquet floor at the orchestra-rail is governed by the stage level, and is generally from 3 ft. 6 ins. to 4 ft. 3 ins. below the stage. A depth of 3 ft. 9 ins. is a good height, as it fixes the eye of the spectator 5 ins. above the stage level."

"The height of the stage, i.e., from the floor to the bottom of the 'gridiron' or rigging-loft, should be 2 or 3 ft. over twice the height of proscenium opening, that the fire-curtain may be raised the full height of the opening." There should be a height of 7 ft. above the gridiron to enable the flymen to adjust their ropes with facility.

Proportioning Gutters and Conductors to Root Surface. — The size of gutters and down-spouts and their distance apart for roofs (of Mill Buildings) with ¼ pitch and of different spans are shown by the following table:*

One half roof span, ft.	10	20	30	40	50	60	70	80
Size of gutter, in.	5	5	6	6	7	7	8	8
" " down-spouts, ins. . . .	3	3	4	4	5	5	6	6
Spacing of down-spouts, ft. . .	50	50	50	50	40	40	40	40

The specifications of the *American Bridge Company* provide as follows for the size of gutters and conductors:†

Span of Roof.	Gutter.	Conductor.
Up to 50 ft.	6 ins.	4 ins. every 40 ft.
50 to 70 "	7 "	5 " " 40 "
70 to 100 "	8 "	5 " " 40 "

Hanging gutters should have a slope of about 1 in. to 16 ft.

"The Produce Exchange Building in New York City, with a roof area of three-fourths of an acre, roughly speaking, has twelve leaders of about 5 ins. diameter. The roof, which is paved with fire-brick, is graded with slopes of perhaps one in fifty toward

* H. G. Tyrrell, C.E.
† M. S. Ketchum, C.E.

the points at which the leader openings are placed, most of these draining surfaces being about 40×70 ft. each. The provision here made is equivalent to about 1 *sq. in. of leader opening to* 140 *sq. ft. of roof surface.* On the Sloane Building, on 19th Street and Broadway, New York City, with a roof area of 18,000 or 20,000 sq. ft., sloping one in twenty-five, there are two leaders of about 6 ins. in diameter, and a third rectangular, 4×6 ins. This gives an allowance of 240 sq. ft. of surface to the square inch of leader opening, while on the Massachusetts Hospital Life Insurance Company's Building, and the Hemenway Building, in Boston, the proportion is only from 60 to 70 sq. ft to the square inch of opening." *

ELEVATORS—SPECIFICATIONS FOR.†

Conditions which should be Considered and made Definite by the Architect, Preliminary to the Elevator Specifications.

(*a*) *The System: Electric or Hydraulic.*—If electric, whether of the drum or friction-drive type. If hydraulic, whether of the horizontal cylinder, the vertical cylinder, or the plunger type.

Where a reliable and sufficient direct-current supply is available for one or two elevators, the electric is unquestionably the best system. For batteries of three or more the system must be determined by the special conditions which exist in every plant, and are relative to the other mechanical equipment, and should be decided only after mature deliberation and consultation with unprejudiced engineers and elevator builders.

(*b*) *Location of Hoistways and Machinery Room.*—The location of the hoistways is rather a matter for the good judgment of the architect, having reference to facility of ingress and egress of passengers, so as to avoid crowding and confusion at the main entrance.

The machinery-room should be immediately adjacent to the hoistways, well ventilated, and protected from dust, large and high enough to permit easy access to all parts of the machines for inspection and repairs.

(*c*) *Number and Sizes.*—The number and sizes will be determined, first, by the space available for hoistways; second, the

* Mr. Dwight Potter in "The Technology Quarterly."

† Prepared by Sydney F. Weston.

number of passengers to be carried during the rush hours; third, the frequency of car departures from the ground floor, or "schedule."

Having determined the square feet of cross-section to be used, the next thing to determine is the number. Three cars, each carrying one-third the passengers, is preferable to two, each carrying one-half, because the car departures are more frequent, reducing the time the passengers must wait at the ground-floor, and therefore lessening the liability of over-congestion and loss of patience by those waiting.

Every machine is likely to need repairs; therefore the more units in the battery the less will the one out of commission be missed. It is far greater economy to have an excess capacity than even a slight under capacity, especially against the time when it may be imperative to shut down one or more machines.

In determining the load for passenger service, allow 80 lbs. per sq. ft. of platform area, and 150 lbs. per passenger.

(*d*) *Loads and Speeds.*—The loads and speeds determine the sizes of machines. The loads having been decided as above, the question of speed is next, and is a most important factor.

Generally the local ordinances limit the car speed, as in New York, to a maximum of 400 ft. per min. for cars that stop at every floor; and to 500 ft. per min. for express cars, those that go the first two-thirds of their travel without stop.

The best elevator insurance companies will not permit electric drum elevators to travel over about 350 ft. per min., whereas the electric friction-drive, or the hydraulic types, are safe and under perfect control for the higher speeds. Four hundred feet per minute is about as high a speed as the human system can stand without unpleasant sensation, and is ample for the best schedules.

In hydraulic systems it is necessary for figuring the pumps and tanks that the maximum number of round trips per hour be specified.

(*e*) *Hoistways.*—The hoistways should be finished to plumb-line dimensions, so that the car running on guide-rails set to plumb-line will at all points have the same clearance.

Provide supports adjacent to the hoistway for the overhead beams at a distance, if possible, of at least 6 ft. from the top of the car frame when the car platform is flush with the top landing, and more is better, in order to have ample "runby," i.e., the distance between the top of the car frame and the lowest

point of the overhead work, so that should the car slide by the top landing a little before the automatic limits shut off the motive power, there would be a minimum danger of running into the overhead work.

For the same purpose there should be a pit at the bottom of the shaft at least 5 ft. deep below the bottom landing.

The distance from the above-mentioned supports for overhead beams, in the clear below the skylight, varies from 4 or 5 to 10 ft., and should be determined by the elevator-maker.

(*f*) *Counterweight, Location of.*—In New York, the Building Department requires that the "counterweights shall be run in a separate shaft from the car, or in a chace separated from the car shaft by a substantial screen or partition for the full height of the hoist." These chaces should be at least 8 ins. deep by 36 ins. long.

(*g*) The Bureau of Buildings for the Borough of Manhattan on April 24, 1902, issued regulations governing the construction, inspection, and operation of passenger elevators, which were published in the "Record and Guide," May 10, 1902, and are especially called to the attention of all architects, as not only obligatory in New York, but excellent practice at all times.

(*h*) The foregoing is intended to give an idea of what the architect must provide in the building for the reception of the elevator apparatus, and what he must determine to enable the maker to intelligently design and lay out his machines.

Above all things, avoid specifying apparatus of special construction. Utilize standard design as much as possible, as, first, it is more apt to be well designed, tested, and built; second, repair parts can be easily and quickly obtained.

Specifications.—These should state:

(1) Kinds of service and number of elevators for each service.

(2) Maximum load. (3) Speed with maximum load. (4) Maximum speed. (5) Load with maximum speed.

(6) Maximum number of round trips per hour for each elevator.

(7) Method of car control.

(8) Sizes of hoistways and area of car platforms.

(9) Travel of car platform in feet; and the number of car landings.

(10) *System.*—If electric, the current and voltage; if hydraulic, the steam pressure for the pumps, or the water pressure if the purchaser provides the pumps, tanks, or other source of water-pressure supply.

(11) An elevation sketch showing landings, supports for overhead beams, space for the overhead sheaves and runbys at top and bottom; a plan sketch showing size and shape of hoistways, entrances, position of car and counterweight guide-rails, and location of space available for machines, pumps, tanks, etc., with reference to the hoistways.

(12) Car and counterweight guide-rails, whether of wood or steel.

(13) Posts or supports for fastening the rails, whether of wood or iron, carried all the way up from the bottom; or iron brackets bolted to the building framework at each floor.

(14) Finished car or cage, value of; i.e., the specified amount to be allowed for each, the design to be subject to the approval of the architect.

(15) Ropes, the number and size of, if not left to the judgment of the maker. Always require the largest sheaves possible, as this factor determines largely the life of the ropes.

(16) Signals, system of; i.e., (*a*) annunciators in the cars with push-buttons at the landings; or (*b*) "up" and "down" signals in the cars, with "up" and "down" buttons at the landings, so arranged that a car going up receives only "up" signals, and a car going down receives only "down" signals, each signal being automatically reset by the first car that passes that floor in the direction for which the signal is given. The latter system adds greatly to the efficiency of a battery of elevators, in that it avoids the confusion of more than one car answering a signal, or a car going in one direction stopping for a passenger going in the opposite direction. Always specify the number of floors at which each car is to land.

(17) *Indicators*, whether at the ground-floor only (for the information of the starter as to the position of the cars) or at all floors.

Indicators are unnecessary with the automatic signals last described, except at the ground-floor, there being at each floor an "up" and a "down" signal to show the first available car in either direction.

(18) Source of power. Specify whether the connection will be brought to the elevator apparatus by the purchaser or by the elevator contractor; if by the latter, give sketch showing the distance, and for the electric system specify whether the wiring is to be open (i.e., on cleats), in moulding, or in conduit; the sizes of wire, and the switches, cutouts, etc.; for an hydraulic system,

the size of pipe for steam supply. Leave the sizes of water-piping to the elevator contractor, and hold him responsible for them.

(19) Pumps and tanks in hydraulic plants to be furnished by the contractor. Specify whether the capacity is to be just ample to do the work, or whether there is to be a reserve capacity, with reserve units, to provide against interfering with the service in case of accident to a pump or tank, but leave the sizes and design to the judgment of a responsible elevator maker.

(20) Foundations for the machine—whether to be provided by the purchaser or by the contractor.

(21) *Miscellaneous.*—Gratings underneath the overhead work, pit-pans, painting in addition to the standard factory finish, and all items not above mentioned, are generally furnished by the purchaser under separate contract, but by whom should be specified in the elevator specifications.

Safety.—Under the subject of "safety" must be considered the most vital feature of the entire apparatus—the mechanical device, or "car safety," for gripping the rails and stopping the car in case the ropes break, or for other reason the car acquires a falling speed in excess of that for which the mechanism is designed.

There are innumerable safeties on the market, but only one or two fulfilling the ideal conditions of first "cushioning," or gradually checking the fall, and then positively and mechanically gripping the rails with a power that increases until the car stops.

Data as to Size and Number of Elevators Required.*—An idea of the practice in elevator installations may be obtained from Table No. 1, which gives the story heights, the approximate area of office space above the first floor, the number of cars, the office area per car, and the area of each car, as actually installed.

To determine the number of trips and car-travel per hour, observations were made at four office buildings in Philadelphia: Drexel Building, ten stories high; Stephen Girard Building, thirteen stories high; Land Title and Trust Building, fifteen stories high; Real Estate Trust Building, seventeen stories high.

In the Drexel Building, which has six elevators, one elevator ran from the first to the fourth story; one elevator from the first to the fifth story; two elevators from the first to the tenth story,

* Extract from a paper by Mr. Charles G. Darrach read before the American Society of Civil Engineers, October 2, 1901, and published in the *American Architect*, October, 1901.

stopping, if required, at any of the floors; and two elevators ran to the fifth story "express," and served all the stories above.

TABLE NO. 1.

Building.	Stories.	Office Area above First Floor, Sq. Ft.	No. of Cars.	Square Feet per Car.	Area of Car, Sq. Ft.
St. Paul Building, New York .	25	83,200	6	13,900	23.6
Empire Building, New York. .	21	150,000	10	15,000	42.0
N. American Building, Philadelphia.	18	90,500	5	18,100	27.6
Real Estate, Philadelphia. . . .	17	155,650	10	15,560	23.7
Bowling Green, New York. . . .	16	222,000	9	24,700	. . . .
Land Title, Philadelphia.	15	66,400	5	13,300	29.6
Stephen Girard, Philadelphia.	13	67,000	4	16,750	29.0
Drexel Building, Philadelphia	10	130,000	6	21,700	21.4

At the Stephen Girard Building there were four elevators, which ran "accommodation" through.

At the Land Title and Trust Company's Building there were five elevators which ran on schedule time, "accommodation" to all floors except the second.

At the Real Estate Building all the cars ran "accommodation" through.

Table No. 2 shows the results obtained.

TABLE NO. 2.

Building.	Stories.	Height, Feet.	Travel per Trip, Feet.	Trips per Hour.	Average Feet per Minute.
Drexel Building.	4	40	80	60	80
" "	5	50	100	52	87
" "	10[1]	108	216	35	126
" "	10[1]	108	216	35	126
Stephen Girard Building.	13	150	300	30	150
Land Title Trust Building. . .	15	180	360	27[1]	162[1]
" " " " . . .	15	180	360	24[2]	144[2]
Real Estate Trust Building. . .	17	200	400	25	167

[1] Actual. [2] Estimated.

From observations at the Drexel Building in 1897, during the noon hour, the up-travel from the first floor reached 800 passengers, with a maximum of 12 to 13 passengers in the car. The cars were also overtaxed all day, from 10 A.M. until 4 P.M.

At the Land Title Building, running twenty-four trips per hour, the service was very satisfactory. There was a slight crowding during the noon hour; this, however, can be remedied by the use of the improved car-signalling apparatus.

At the Stephen Girard Building, the cars are crowded during the noon hour, and also between three and five in the afternoon.

All three of these buildings are well filled.

The Real Estate Building was not fully occupied, so that the elevator service there could not be fairly judged.

Using the trips per hour as observed and estimated, and equating the car area by the formula:

$$a = \frac{A}{T \times 22},$$

in which a = square feet of car area;
A = square feet of office area;
T = total trips per hour.

we derive the following table:

TABLE NO. 3.—EQUATED CAR AREAS.

Building.	Stories.	Number of Cars.	Sq. Ft. of Office Area per Car.	Actual Car Area in Sq. Ft.	Equated Car Area in Sq. Ft.	Estimated Trips per Hour.
St. Paul	25	6	13,900	23.6	31.5	120
Empire........	21	10	15,000	42.0	31.0	220
Real Estate. ...	17	10	15,560	23.7	28.3	250
Bowling Green.	16	9	24,700	....	43.1	234
Land Title.....	15	5	13,300	....	22.4	135
" "	15	5	13,300	29.6	25	120
Stephen Girard.	13	4	16,750	23	25.4	120
Drexel........	10	6	21,700	21.4	28.2	210

At the Drexel Building the two cars which ran "express" to the fifth floor, and the two cars which ran "accommodation" through, made the same number of trips, and carried practically the same number of passengers. It would be interesting to know whether similar results are obtained in any other buildings, and the advantage gained in arranging the travel of the various cars to serve "accommodation" through the entire trip, "express" part way, or providing separate service to different heights in the building.

Using the same formula, and equating to obtain the square feet of office space per car, with given-sized cars, and the number of through trips heretofore used, we have the following results, with cars of 25 and 30 sq. ft. area:

TABLE NO. 4.—SQUARE FEET OF OFFICE AREA PER CAR.

Stories.	Car Area = 25 sq. ft.	Car Area = 30 sq. ft.
25	11,000 sq. ft. per car	13,200 sq. ft. per car
21	12,100 " " "	14,500 " " "
17	13,750 " " "	16,500 " " "
16	14,300 " " "	17,160 " " "
15	14,850 " " "	17,820 " " "
13	16,500 " " "	19,800 " " "
10	19,250 " " "	23,100 " " "

Additional data furnished by Mr. Kloman of the Otis Elevator Co:

Name of Building, all in New York City.	No. of Elevators.	No. of Floors.	Total Floor Area.	Floor Area per Elevator.
Broad Exchange	18	20	465,540	25,864
Park Row (Ivins Syndicate)	10	25	315,000	31,500
Atlantic Mutual	6	18	162,000	27,000
American Exchange Bank	3	16	72,000	24.000
Bank of Commerce	7	19	172,000	24,571
S.E. cor. Broadway and Maiden Lane	6	18	129,000	21,500
Empire Building	10	20	170,000	17,000

The Empire Building is said to be noted for its quick service, and the Park Row Building for slowness.

According to Mr. Kloman, the officers of the Otis Elevator Co. have come to the conclusion that the best service is obtained with a large number of small cars having a capacity of not over 15 passengers, rather than with fewer large cars.

Electric Elevator with Push-button Control.—This is perhaps the most important of the latest improvements in elevators, as it permits of the installation of elevators in residences and other buildings where a constant attendant would be both expensive and undesirable. This type of elevator is particularly adapted to private residences, apartment houses, hospitals, and other places where the service is intermittent and it is desired to do away with the expense of an attendant.

"The elevator is always ready for service, and it is equipped with every safeguard which human ingenuity can devise against the possibility of accident."

The operation of the elevator is as follows: A passenger desiring to use the elevator presses a button placed near the elevator shaft, and the car, if not in use, immediately travels to that floor and stops automatically. When the car has come to rest at that

floor, the door can be opened. The passenger then enters the car and closes the door. The car will not leave that floor unless the door is tightly closed. Inside the car there is a series of push-buttons, numbered to correspond with the various floors. The

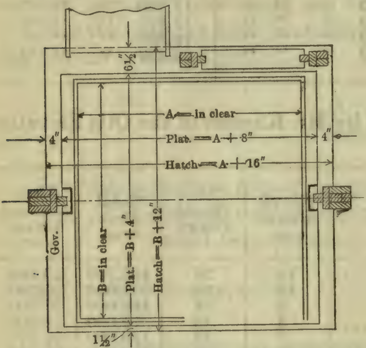


FIG. A. WOOD GUIDES. SIDE POST. MACHINE AND COUNTERWEIGHT AT BACK OR MACHINE OVERHEAD.

With machine and counterweight at side, the width of hatch must be increased 3 ins., and the depth may be 5 ins. less. Steel guides effect a saving in width of hatch of 1 to 2 ins.

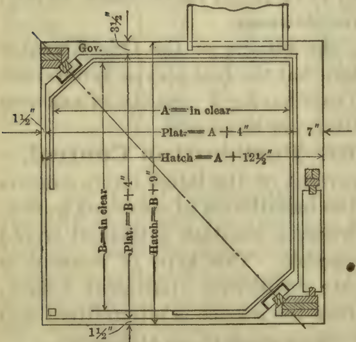


FIG. B. WOOD GUIDES. CORNER POST. MACHINE AT BACK OR OVERHEAD. COUNTERWEIGHT AT SIDE.

With steel corner guides, depth of hatch is same as in Fig. B, but width is ½ in. less.

passenger pushes the proper button and the car proceeds to the desired landing and stops automatically. Not until the passenger has left the car and closed the door can the elevator be controlled from any other floor. Should the passenger desire, for any reason, to stop the car at any point of its travel, he can do

so instantaneously, by merely pushing the safety button with which the car is provided.

Standard Relations of Hatchway, Platform, and Car Sizes.—In their 1903 Catalogue the Otis Elevator Co. (which furnishes a large proportion of the elevators in the United States) published sixteen engravings showing the required size of hatchway and car platforms under different conditions, taking the inside dimensions of the car as a base.

As these relative dimensions apply to all elevators—electric, hydraulic, steam, and belt-driven—they will be found very useful for reference in the preparation of plans.

The diagrams for two of the most common installations are reproduced on opposite page.

MAIL CHUTES.

The Cutler patent system of mailing letters from each floor, by means of a specially constructed chute connected with the receiving-box at the bottom, has come into such general use in public buildings, apartment houses, and hotels that architects should be informed in regard to the restrictions affecting the same and what is required in the way of preparation. The system is installed by the patentees, under regulations of the Post Office Department governing its construction and location. It may be placed in any building of more than one story used by the public, where there is free delivery and collection service, in the discretion of the local postmaster, subject to whose approval the contracts are made.

The chute must extend in a vertical line, must be exposed to view, and accessible throughout its entire length. It is made in removable sections, to facilitate clearing it in the event of accident.

The requirements for "preparatory work" are described in Part II, Building Construction and Superintendence, p. 520. Before the final completion of plans, however, architects or owners should submit the same to the Cutler Mfg. Co., Rochester, N. Y., with whom contracts for the installation must be made. The whole apparatus, when erected and the Government lock put on the box, passes under the exclusive care and control of the Post Office Department.

REFRIGERATORS.

The following information is given as a guide to architects in providing for refrigerators in fine residences, hotels, club buildings, etc.

A consultation with some reliable refrigerator builder,* however, is always wise before deciding in relation to space to be occupied by refrigerators, refrigerating rooms, freezers, etc., as *a satisfactory refrigerator cannot be adapted to a badly proportioned space.* Care should be taken to select a refrigerator simple in its working and *easily cleansed,* as modern sanitary science has traced much sickness to poor refrigeration. *Thorough insulation* is one of the most important features in a refrigerator, as upon this depends economy in the use of ice, the keeping of the cold air, and the consequent perfect preservation of the food.

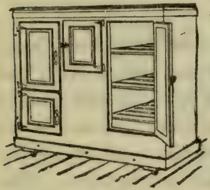


Fig. 1

Fig. 1 is a kitchen refrigerator for use in families of ordinary size, and has the ice located in the centre. Depth should not be over 3 ft. nor under 2 ft. Height may be 4 to 7 ft. Length of front largely determines the capacity, and should be, say, from 5 to 7 ft.

Fig. 2 shows greater capacity, and is better adapted for use in large families, entertaining considerably, and for small clubs, boarding-houses, restaurants, private hospitals, etc. This style is known as a "combination" refrigerator, from the fact that it contains separate compartments for the various kinds of food. The large compartment at the left is specially for large meats, and packages in bulk, and is fitted with shelves and meat-hooks. The right end of the refrigerator is divided by a partition into two compartments, the drawers being for steaks, chops, jellies, etc., and the door above

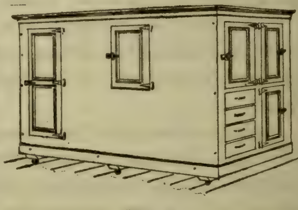


Fig. 2

* The leading builders of high-class refrigerators are: The Lorillard Refrigerator Co., New York; McCray Refrigerator Co., Kendallville, Ind.; Monroe Refrigerator Co., Lockland, Ohio; Wickes Refrigerator Co., Chicago, Ill.

for vegetables and sundries. The compartment to the right of this is specially for milk and butter, and should be absolutely separate from all other compartments. One ice-tank supplies cold air to all compartments, and is filled through a door in the front.

A convenient arrangement is a window in the wall at back of refrigerator, through which ice may be passed into refrigerator.

Refrigerators over two feet in depth should be built in sections bolted together, rendering them easy to transport and handle in contracted space.

Fig. 3 is a refrigerator for use in butler's pantries, where economy of space is important. The ice-tank is arranged to come out on a runway, for convenience in filling. When the ice-tank is pushed back, this runway folds up, and an outside door closes over it. This does away with the necessity of cutting through the counter-top, and permits the ice-tank to be readily taken out for cleansing purposes. The height should be about 2 ft. 8 ins., depth about 2 ft. Length of front determines capacity, but should never be less than 2 ft. 10 ins. In every 3 ft or 3 ft. 6 ins. one ice-tank is allowed. The finish, wood, trim, and hardware should correspond with other fittings.

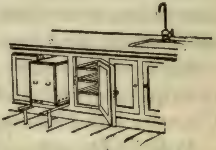


Fig. 3

Drainage.—A *short, accessible*, well-trapped drain is imperative, and should be as nearly under the centre of the ice compartment as possible. It is well to have refrigerators on casters, so they can be easily moved for cleaning about them.

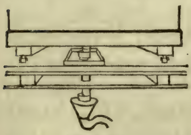


Fig. 4

Fig. 4 shows a good drainage arrangement, permitting removal of refrigerator at will.

Plumber's pan for reception of refrigerator drip should be countersunk in floor.

Where a very low temperature is required, as for game or fish carried in large quantities, or in medical colleges where the object is to preserve bodies, it is absolutely necessary that ice should go into the tanks from top.

Usual complement of refrigerators for use in ordinary families: one in kitchen; one in butler's pantry. Large families same, with greater capacity. Small clubs, small restaurants, etc., one general storage; one wine; one in or near kitchen, for cook's

use; one fish. Large hotels, clubs, restaurants, etc.: one storage for large meat; one in or near kitchen, for cook's use; one fish; one milk and butter; one in storeroom; one ice-cream (in hotels); one wine. Private hospitals: one large storage; one for cook's use, in or near kitchen; one for milk and butter; one iron-lined box for broken ice. Large hospitals same, but increased capacity, and a small refrigerator in each ward. Isolated hospitals should have large storage ice-houses in addition. Medical colleges, for preserving bodies, with accommodations for eight bodies: dimensions about 8 ft. 6 ins. front, 7 ft. 6 ins. deep, and 9 ft. high. Ice going into tanks from top.

Revolving Doors.—A great improvement over the ordinary doors, or storm-doors, for many purposes. For description see Part II, Building Construction and Superintendence.

Tower Clocks—Dimensions of Clock Faces.—For description of requirements of installation of tower clocks, see *Churches and Chapels*, p. 154.

Rule for Diameter of Dials.—"To look well and show plainly, dials should be 1 ft. diameter for every 10 ft. of elevation and should set out flush with or close to the line of the building or tower." *

DIMENSIONS OF SOME LARGE CLOCK FACES.

Tower Clock, Depot of the Central Railroad of New Jersey, at Communipaw.—Diameter of single dial, 14 ft. 3 ins.; minute hand is 7 ft. long, weighs 40 lbs.; hour hand is 5 ft. long, weighs 28 lbs. The motive power is furnished by a weight of 700 lbs., hung from a $\frac{3}{8}$-in. steel cable.

Four-dial Clock, New York Produce Exchange.—Diameter of each dial, 12 ft. 6 ins.

Four-dial Clock, Chronicle Tower, San Francisco.—Diameter of each dial, 16 ft. 6 ins.; length of minute hands, 8 ft.; length of hour hands, 5 ft. 6 ins. The mechanism of the clock is 6 ft. 1 in. high and weighs 3,000 lbs.

Pneumatic Clock, City Hall and Court House, Minneapolis.—Dials, 23 ft. 4 ins. in diam.

LIBRARY STACKS—CAPACITY OF SHELVING.

General Description of the Library Stack System, using Iron or Steel Stacks.—The unit of the system is the *shelf compartment*, or the space between two adjacent

* Seth Thomas Clock Co.

partitions or *shelf supports*. A row of *compartments*, side by side, constitutes a *range*. A number of *ranges* form a *stack*.

When the *compartments* are placed against walls, and are accessible from only one side, they are called *single-faced;* and when placed free from walls, thus accessible on both sides, they are *double-faced*.

A standard *single-faced compartment* is 3 ft. long, 8 or 10 ins. wide, and 7 ft. high. A standard *double-faced compartment* is 16 or 20 ins. wide, other dimensions the same. Standard *shelves* are 3 ft. long and 8 or 10 ins. wide. The aisles between the ranges vary from 2 ft. 8 ins. to 3 ft. 4 ins. in width.

The *shelf supports* are made in various ways, differing with each manufacturer. They should not seriously break the smooth surface at the side, or expose the last book to damage. The *shelves* are usually constructed of steel. The weight of *stacks* and *shelves* (as made by the Snead & Co. Iron Works) with their load of *books* is about 30 lbs. per cubic foot of *stack*.

When there are upper floors they are usually referred to as *decks;* the height from *deck* to *deck* varies from 7 ft. to 7 ft. 6 ins., 7 ft. being the standard. The *deck framing* consists of steel tees, angles, and bars, and the *floor covering* is of marble or rough plate glass. The weight of *deck framing* and *floor covering* is about 24 lbs. per sq. ft. for marble and 18 lbs. for glass.

Capacity of Library Shelving.—The *capacity* of a library depends upon its character; for an ordinary circulating library the *capacity* is about ten volumes per lineal foot of shelf; for the Library of Congress it is about eight and a half volumes per lineal foot, and this is about the average for an ordinary collection of books.

The number of books a room will hold may be estimated as follows:

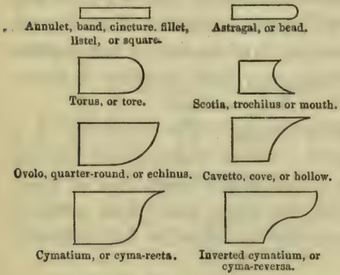
Let us suppose that the cases are 7 ft high, 16 ins. deep, and have books on each side; that the width of passageway between the cases is the minimum of 32 ins.; and that each shelf is 36 ins. in length. The floor-space that one division of one side of the case will take is half the width of the case (8 ins.) plus half the width of passageway (16 ins.), multiplied by the length of the shelf (36 ins.), which gives a result of 6 sq. ft. If the average number of shelves in the division is 7, and there are 8½ books to the foot, the capacity of the division is 180 volumes, or an average of 30 books to the square foot of floor area. In this calculation no account has been taken of the stairs, windows,

doors, or cross gangway, and only a minimum width of passageway has been allowed. If space for these is taken into consideration, a conservative estimate of shelving capacity of a room will work out at about 22 volumes to the square foot, for each deck or story 7 ft. high.

In the Congressional Library at Washington the double-faced compartments are 24 ins. wide over all, and the aisles between 3 ft. 4 ins. wide, making 5 ft. 4 ins. between centres of compartments. The stack rooms are nine stories high, each story being 7 ft. from floor to floor. The floors proper are thin white-marble slabs, with polished side down for reflecting the light.

CLASSICAL MOULDINGS.*

Mouldings are so called because they are of the same shape throughout their length as though the whole had been cast in the same mould or form. The regular mouldings, as found in remains of classic architecture, are eight in number, and are known by the following names:



The last two are both called "ogee."

Some of these terms are derived thus: Fillet, from the French word *fil*, "thread"; astragal, from *astragalos*, "a bone of the heel," or "the curvature of the heel"; bead, because this moulding, when properly carved, resembles a string of beads; torus, or tore, the Greek for *rope*, which it resembles when on the base of a column; scotia, from *skotia*, "darkness," because of the strong shadow which its depth produces, and which is increased by the projection of the torus above it; ovolo, from *ovum*, "an egg," which this member resembles when carved, as in the Ionic capital; cavetto, from *cavus*, "hollow"; cymatium, from *kumaton*, "a wave."

Characteristics of Mouldings.—Neither of these mouldings is peculiar to any one of the orders of architecture; and although each has its appropriate use, yet it is by no means confined to any certain position in an assemblage of mouldings.

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* See also Glossary, under Moulding.

The use of the fillet is to bind the parts, as also that of the astragal and torus, which resemble ropes. The ovolo and cyma-reversa are strong at their upper extremities, and are therefore used to support projecting parts above them.

The cyma-recta and cavetto, being weak at their upper extremities, are not used as supporters, but are placed uppermost to cover and shelter the upper parts. The scotia is introduced in the base of a column to separate the upper and lower torus, and to produce a pleasing variety and relief.

The form of the bead and that of the torus are the same; the reasons for giving distinct names to them are that the torus, in every order, is always considerably larger than the bead, and is placed among the base mouldings, whereas the bead is never placed there, but on the capital or entablature. The torus, also, is seldom carved, whereas the bead is; and while the torus, among the Greeks, is frequently elliptical in its form, the bead retains its circular shape. While the scotia is the reverse of the torus, the cavetto is the reverse of the ovolo, and the cyma-recta and cyma-reversa are combinations of the ovolo and cavetto.

THE CLASSICAL ORDERS.*

"In the classical styles several varieties of column and entablature are in use. These are called the orders. Each order comprises a column with a base, shaft, and capital, with or without a pedestal, with its base, die, and cap, and is crowned by an entablature, consisting of architrave, frieze, and cornice. The entablature is generally about one fourth as high as the column, and the pedestal one third, more or less.

"Among the Greeks the forms used by the Doric race, which inhabited Greece itself and had colonies in Sicily and Italy, were much unlike those of the Ionic race, which inhabited the western coast of Asia Minor, and whose art was greatly influenced by that of Assyria and Persia. Besides the Ionic and Doric styles, the Romans devised a third, which employed brackets, called modillions, in the cornice, and was much more elaborate than either of them; this they called the Corinthian.

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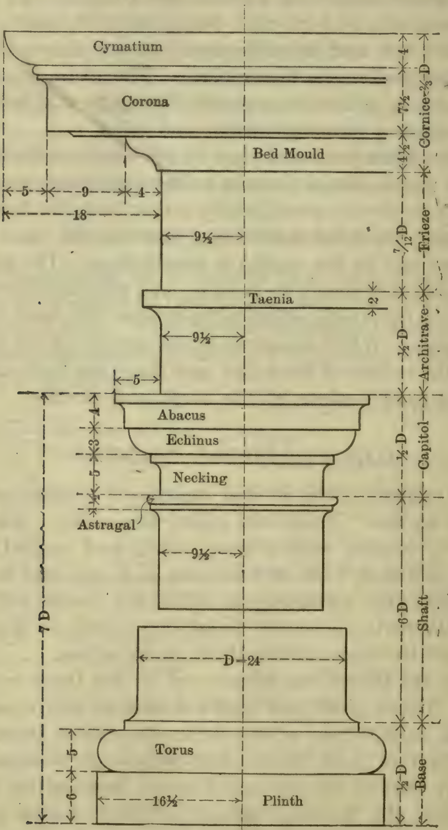
* The paragraphs in quotation-marks are taken from "The American Vignola" by Prof. Wm. R. Ware, by permission of the owners of the copyright, the International Text-book Company, proprietors of the International Correspondence Schools. The engravings were made especially for this book, and correspond with the original drawings prepared by Giacomo Barozzi da Vignola.

They used also a simple Doric called the Tuscan, and a cross between the Corinthian and Ionic called the Composite. These are the *five orders*. The ancient examples vary much among



Dimensions are in 24ths of Diameter.

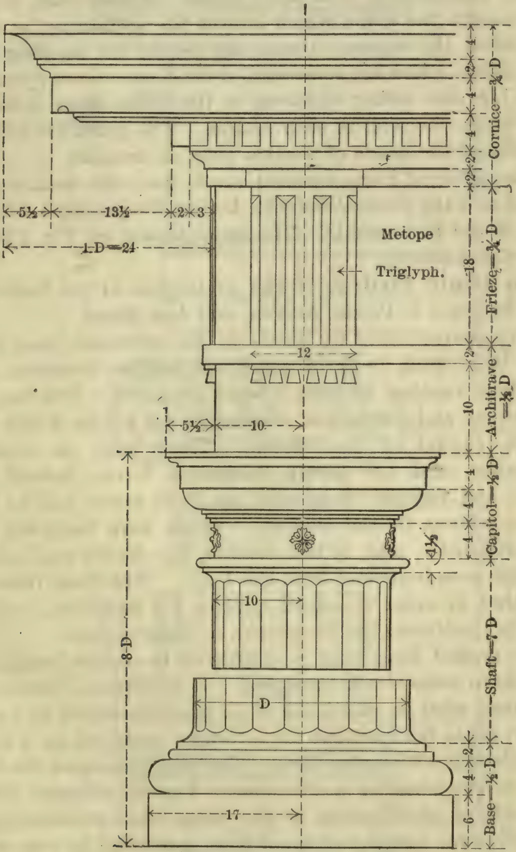
Fig. 1

The Tuscan Order.

themselves and differ in different places, and in modern times still further varieties are found in Italy, Spain, France, Germany, and England. The best known and most admired forms for the orders are those worked out by Giacomo Barozzi da

Vignola in the sixteenth century from the study of ancient examples."

The Tuscan Order.—"The distinguishing characteristic of the Tuscan order is simplicity. Any forms of pedestal,



Dimensions are in 24ths of Diameter

Fig. 2

The Doric Order.

column, and entablature that show but few mouldings, and those plain, are considered to be Tuscan."

The Doric Order.—"The distinguishing characteristics of the Doric order are features in the frieze and in the bed-mould above it called triglyphs and mutules, which are supposed to be derived from the ends of beams and rafters in a primitive wooden construction with large beams. Under each triglyph, and beneath the tænia which crowns the architrave, is a little fillet called the regula. Under the regula are six long drops, called guttæ, which are sometimes conical, sometimes pyramidal. There are also either eighteen or thirty-six short cylindrical guttæ under the soffit of each mutule. The guttæ are supposed to represent the heads of wooden pins, or treenails.

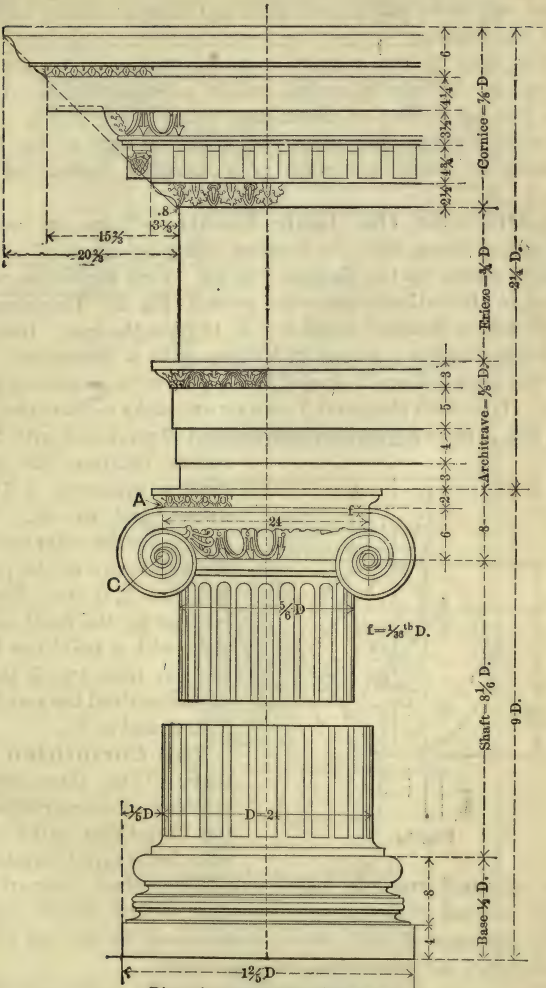
"Two different Doric cornices are in use—the *mutulary* with bracket and the *denticulated* with dentils, the principal difference being in the bed-mould." The order shown by Fig. 2 has the denticulated cornice.

The Ionic Order.—"The prototypes of the Ionic order are to be found in Persia, Assyria, and Asia Minor.

"It is characterized by bands in the architrave and dentils in the bed-mould, both of which are held to represent small sticks laid together to form a beam or a roof. But the most conspicuous and distinctive feature is the scrolls which decorate the capital of the column. These have no structural significance, and are purely decorative forms derived from Assyria and Egypt. Originally the Ionic order had no frieze and no echinus in the capital. These were borrowed from the Doric order, and, in like manner, the dentils and bands in the Doric were borrowed from the Ionic. The Ionic frieze was introduced in order to afford a place for sculpture, and was called by the Greeks the Zoophorus, or Figure-bearer.

"The typical Ionic base is considered to consist mainly of a scotia, as in some Greek examples. It is common, however, to use instead what is called the Attic base, consisting of a scotia and two fillets between two large toruses, mounted on a plinth, the whole half a diameter high. The plinth occupies the lower third, or one sixth of a diameter. Vignola adopted for his Ionic order a modification of the Attic base, substituting for the single large scotia two small ones, separated by one or two beads and fillets, and omitting the lower torus." This is the base shown in the engraving.

"The Ionic frieze is plain, except for the sculpture upon it. It sometimes has a curved outline, as if ready to be carved, and



Dimensions are in 24ths of Diameter.

Fig. 3
The Ionic Order.

is then said to be pulvinated, from pulvinar, a bolster, which it much resembles.

"The shaft of the column is ornamented with twenty-four flutings, semicircular in section, which are separated not by an arris, but by a fillet of about one fourth their width. This makes the flutings only about two thirds as wide as the Doric channels, or about one-ninth of a diameter, instead of one sixth."

To Describe the Ionic Volute.—There are several methods of doing this, the simplest being by means of centres found as shown by the diagram Fig. 3*a*. First locate the centre of the eye $\frac{1}{4}D$ vertically below the point A, Fig. 3. Then describe a circle with a diameter equal to $\frac{1}{18}D$, to form the eye. Inside of this circle inscribe a square at 45 degrees to a horizontal; then draw the axes 1–3 and 2–4, and divide each of these into six equal parts. Then with the point 1 as a centre, and a radius extending to A, Fig. 3, draw a quarter-circle to line 1–2 produced, with 2 as a centre, continue the curve until it intersects 2–3 produced, and so on. The centres for the outer curve of the volute are at the points 1, 2, 3, 4, 5, 6, etc. For the centres for the inner curve, start with a point one third the way from 1 to 5, then a point one third the way from 2 to 6, and so on.

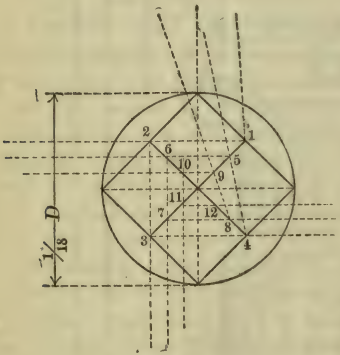
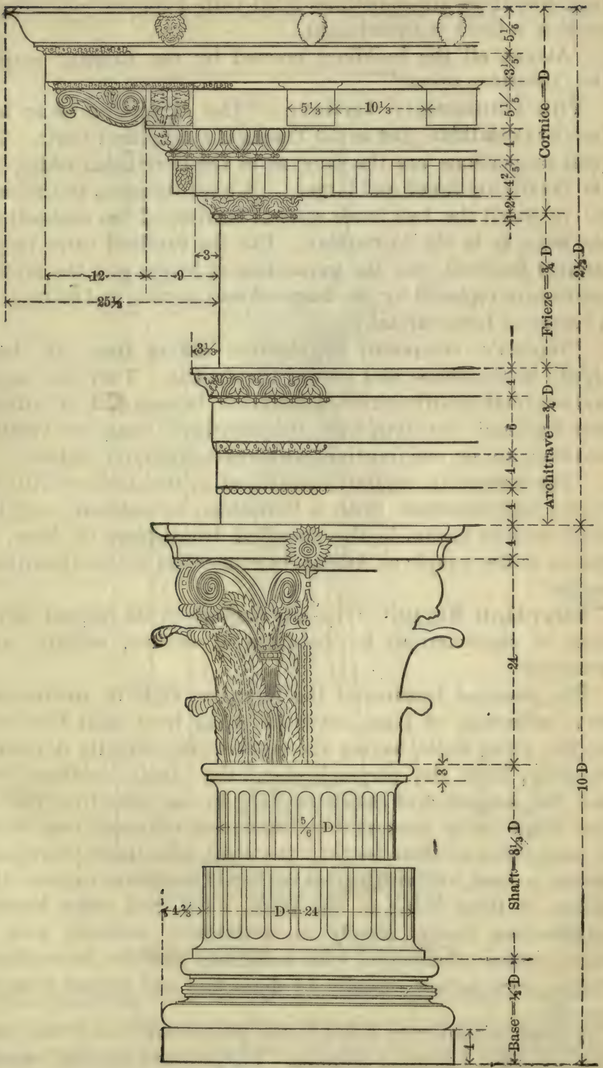


Fig. 3a

The Corinthian Order.—"The three distinguishing characteristics of the Corinthian order are a tall, bell-shaped capital, a series of small brackets called modillions, which support the cornice instead of mutules, in addition to the dentils, and a general richness of detail which is enhanced by the use of the acanthus leaf in both capitals and modillions.

"Here, again, the Attic base is commonly used, but sometimes, especially in large columns, a base is used which resembles Vignola's Ionic base, except that it has two beads between the scotias instead of one, and also a lower torus. The shaft is fluted like the Ionic shaft, with twenty-four semicircular flut-



Dimensions are in 24ths of Diameter

THE CORINTHIAN ORDER

Fig. 4

ings, but these are sometimes filled with a convex moulding or cable to a third of their height.

"Almost all the buildings erected by the Romans employ the Corinthian order."

The Composite Order.—"The Composite order is a heavier Corinthian, just as the Tuscan is a simplified Doric. The chief proportions are the same as in the Corinthian order, but the details are fewer and larger. It owes its name to the capital, in which the two lower rows of leaves and the caulicoli are the same as in the Corinthian. But the caulicoli carry only a stunted leaf-bud, and the upper row of leaves and the sixteen volutes are replaced by the large echinus, scrolls, and astragal of a complete Ionic capital.

"Vignola's composite entablature differs from his Ionic chiefly in the shape and size of the dentils. They are larger, and are more nearly square in elevation, being a fifth of a diameter high and one sixth wide, the interdentil being one twelfth, and they are set one fourth of a diameter apart, on centres.

"The composite capital is employed in the Arch of Titus in Rome, and elsewhere, with a Corinthian entablature, and the block cornice occurs in the so-called frontispiece of Nero, as well as in the temple at Athens, in connection with a Corinthian capital."

Egyptian Style.*—The architecture of the ancient Egyptians is characterized by boldness of outline, solidity, and grandeur.

The principal features of the Egyptian style of architecture are: uniformity of plan, never deviating from right lines and angles; thick walls, having the outer surface slightly deviating inwardly from the perpendicular; the whole building low; roof flat, composed of stones reaching in one piece from pier to pier, these being supported by enormous columns, very stout in proportion to their height; the shaft sometimes polygonal, having no base, but with a great variety of handsome capitals the foliage of these being of the palm, lotus, and other leaves; entablatures having simply an architrave, crowned with a huge cavetto ornamented with sculpture; and the intercolumniation very narrow, usually $1\frac{1}{2}$ diameters and seldom exceeding $2\frac{1}{2}$.

A great dissimilarity exists in the proportion, form, and general feature of Egyptian columns. For practical use the column

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* From "The American House Carpenter," by R. G. Hatfield.

shown in Fig. 5 may be taken as a standard of the Egyptian style.

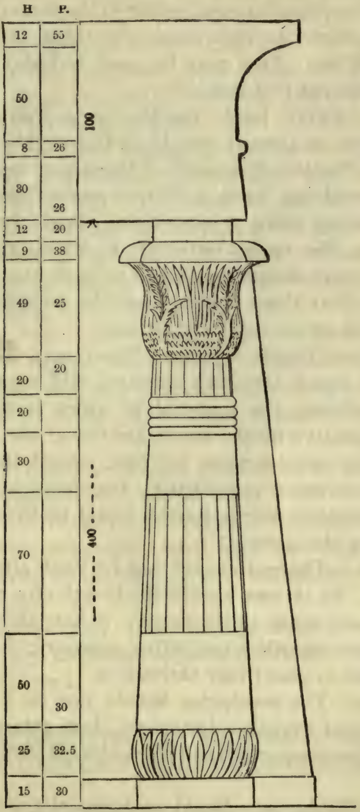

FIG. 5.—EGYPTIAN ARCHITECTURE.
(Diameter divided into 60 parts.)

LIGHTNING CONDUCTORS.

The following rules for the erection of lightning conductors were issued in 1882 by the Explosive Department of the English Home Office to the occupiers of all factories and magazines for explosives, and to those local and police authorities upon whom devolves the inspection of stores of explosives:

1. *Material of Rod.*—Copper, weighing not less than 6 oz. per foot run, the electrical conductivity of which is not less than 90 per cent. of that of pure copper, either in the form of rod, tape, or rope of stout wires, no individual wire being less than No. 12 B. W. G. (.109 in.). Iron may be used, but should not weigh less than 2½ lbs. per foot run.

2. *Joints.* — Every joint, besides being well cleaned and screwed, scarfed, or riveted, should be thoroughly soldered.

3. *Form of Points.*—The point of the upper terminal * of the conductor should not have a sharper angle than 90°. A foot below the extreme point a copper ring should be screwed and soldered on to the upper terminal, in which ring should be fixed three or four sharp copper points, each about 6 ins. long. It is desirable that these points should be so platinized, gilded, or nickel-plated as to resist oxidation.

4. *Number* and *Height of Upper Terminals.*—The number of conductors or upper terminals required will depend upon the size of the building, the material of which it is constructed, and the comparative height above ground of the several parts. No general rule can be given for this, except that it may be assumed that the space protected by the conductor is, as a rule, a cone, the radius of whose base is equal to the height of the conductor from the ground.

5. *Curvature.*—The rod should not be bent abruptly around sharp corners. In no case should the length of a curve be more than half as long again as its chord. A hole should be drilled in string-courses or other projecting masonry, when possible, to allow the rod to pass freely through it.

6. *Insulators.*—The conductor should not be kept from the building by glass or other insulators, but attached to it by fastenings of the same metal as the conductor itself is composed of.

7. *Fixing.*—Conductors should preferentially be taken down the side of the building which is most exposed to rain. They should be held firmly, but the holdfasts should not be driven in so tightly as to pinch the conductor or prevent contraction and expansion due to change of temperature.

8. *Other Metal Work.*—All metallic spouts, gutters, iron doors, and other masses of metal about the building should be electrically connected with the conductor.

* The upper terminal is that portion of the conductor which is between the top of the edifice and the point of the conductor.

9. *Earth Connection.*—It is most desirable that, whenever possible, the lower extremity of the conductor should be buried in permanently damp soil. Hence, proximity to rain-water pipes and to drains or other water is desirable. It is a very good plan to bifurcate the conductor close below the surface of the ground, and to adopt two of the following methods for securing the escape of the lightning into the earth: (1) A strip of copper tape may be led from the bottom of the rod to a gas or water main (not merely to a leaden pipe), if such exist near enough, and be soldered to it; (2) a tape may be soldered to a sheet of copper, 3 ft. $\times$ 3 ft. $\times \frac{1}{16}$ in. thick, buried in permanently wet earth and surrounded by cinders or coke; (3) many yards of copper tape may be laid in a trench filled with coke, having not less than 18 sq. ft. of copper exposed.

10. *Protection from Theft, etc.*—In places where there is any likelihood of the copper being stolen or injured, it should be protected by being enclosed in an iron gas-pipe, reaching 10 ft. (if there is room) above ground and some distance into the ground.

11. *Painting.*—Iron conductors, galvanized or not, should be painted. It is optional with copper ones.

12. *Inspection.*—When the conductor is finally fixed it should in all cases be examined and tested by a qualified person, and this should be done in the case of new buildings after all work on them is finished.

Periodical examination and testing, should opportunities offer, are also very desirable, especially when iron earth connections are employed.

ADHESIVE STRENGTH OF SULPHUR, LEAD, AND PORTLAND CEMENT FOR ANCHORING BOLTS.

The following test of these materials is reported in the *American Architect*, page 105, vol. xxiv.:

"Fourteen holes were drilled in a ledge of solid limestone, seven of them being $1\frac{3}{8}$ ins. in diameter and seven of them $1\frac{5}{8}$ ins. in diameter, all being $3\frac{1}{2}$ ft. deep. Seven $\frac{3}{4}$-in. and seven 1-in. bolts were prepared with thread and nut on one end and plain at the other end but ragged for a length of $2\frac{1}{2}$ ft. from the blank end.

"Four were anchored with sulphur, four with lead, and six with cement, mixed neat. Half of each were $\frac{3}{4}$-in. and half 1-in.

bolts, and all of them were allowed to stand till the cement was two weeks old. At the expiration of this time a lever of sufficient power was rigged and all the bolts were pulled with the following result:

"*Sulphur.*—Three bolts out of four developed their full strength, 16,000 and 31,000 lbs. One 1-in. bolt failed by drawing out under 12,000 lbs.

"*Lead.*—Three bolts out of four developed their full strength, as above; one 1-in. bolt pulled out under 13,000 lbs.

"*Cement.*—Five of the bolts out of six broke without pulling out; one 1-in. bolt began to yield in the cement at 26,000 lbs., but sustained the load a few seconds before it broke.

"While this experiment demonstrated the superiority of cement, both as to strength and ease of application, yet it did not give the strength per square inch of area. To determine this, four specimens of limestone were prepared, each 10 ins. wide, 18 ins. long, and 12 ins. thick, two of them having $1\frac{3}{4}$-in. holes, and two of them $2\frac{3}{4}$-in. holes drilled in them. Into the small holes 1-in. bolts were cemented, one of them being perfectly plain round iron, and the other having a thread cut on the portion which was imbedded in the cement. Into the $2\frac{3}{4}$-in. holes were cemented 2-in. bolts similarly treated, and the four specimens were allowed to stand thirteen days before completing the experiment. At the end of this time they were put into a standard testing-machine and pulled. The plain 1-in. bolt began to yield at 20,000 lbs., and the threaded one at 21,000 lbs. The 2-in. plain bolt began to yield at 34,000 lbs., and the threaded one at 32,000 lbs., the strain in all cases being very slowly applied. The pump was then run at a greater speed, and the stones holding the 2-in. bolts split at 67,000 lbs. in the case of the smooth one and at 50,000 lbs. in the case of the threaded one.

"It is thus seen that cement is more reliable, stronger, and easier of application than either lead or sulphur, and that its resistance is from 400 to 500 lbs. per square inch of surface exposed. It is also a well-ascertained fact that it preserves iron rather than corrodes it. The cement used throughout the experiment was an English Portland cement."

EFFLORESCENCE ON BRICKWORK.

There are at least three different substances which may cause the white efflorescence often seen on the face of brickwork.

Of these, carbonate of soda is the most common upon new work, after the lime stains have been removed. This is due to the action of the lime mortar upon the silicate of soda in the bricks. Silicate of soda seldom occurs in brick unless the clay used is a salt clay.

The only other white efflorescence of importance is chiefly composed of sulphate of magnesia. This is due to pyrites in the clay, which, when burned, gives rise to sulphuric acid, and the latter unites with magnesia in the lime mortar.

The above are the results of actual examinations by Mr. Samuel Cabot, chemist. The conclusions arrived at are these:

I. The efflorescence is never due to the bricks alone, and seldom to the lime alone.

II. To avoid it, the bricks should be covered with an oily preservative capable of keeping the salts from exuding. Linseed oil cannot fill the requirements, as it is injured by the mortar.

RELATIVE HARDNESS OF WOODS.

Taking shell-bark hickory as the highest standard of our forest-trees, and calling that 100, other trees will compare with it for hardness as follows:

Shell-bark hickory	100	Yellow oak	60
Pignut hickory	96	Hard maple	56
White oak	84	White elm	58
White ash	77	Red cedar	56
Dogwood	75	Wild cherry	55
Scrub oak	73	Yellow pine	54
White hazel	72	Chestnut	52
Apple-tree	70	Yellow poplar	51
Red oak	69	Butternut	43
White beech	65	White birch	43
Black walnut	65	White pine	30
Black birch	62		

WEIGHT OF ROUGH LUMBER PER 1,000 FEET.

BOARD MEASURE (APPROXIMATE).

(For weight of various woods see table, pp. 1341 to 1344.)

	Dry.	Partly Seasoned.	Green.
Pine and hemlock	2,500 lbs.	2,700 lbs.	3,000 lbs.
Norway and yellow pine	3,000 "	4,000 "	5,000 "
Oak and walnut	4,000 "	5,000 "	
Ash and maple	3,500 "	4,000 "	

FORCE OF THE WIND.

According to experiments made in 1890 or thereabouts, by Asst. Prof. C. F. Marvin, U. S. Signal Service, the relation between wind pressure and velocity is given very accurately by the formula $p = .004V^2$, where p = pressure in pounds per square foot on a flat surface normal to the direction of the wind, and V denotes velocity in miles per hour. Smeaton considered the pressure as equal to $.005V^2$.

The following table based on Marvin's formula is quoted by Profs. Turneaure and Ketchum. See also Trautwine's Pocket-book, p. 321, note.

Miles per Hour.	Feet per Minute.	Feet per Second.	Force, in Pounds, per Square Foot.	Description.
1	88	1.47	0.004	Hardly perceptible
2	176	2.93	0.014	Just perceptible
3	264	4.4	0.036	
4	352	5.87	0.064	Gentle breeze
5	440	7.33	0.1	
10	880	14.67	0.4	Pleasant breeze
15	1,320	22	0.9	
20	1,760	29.3	1.6	Brisk gale
25	2,200	26.6	2.5	
30	2,640	44	3.6	High wind
35	3,080	51.3	4.9	
40	3,520	58.6	6.4	Very high wind
45	3,960	66	8.1	
50	4,400	73.3	10.0	Storm
60	5,280	88	14.4	Great storm
70	6,160	102.7	19.6	
80	7,040	117.3	25.6	Hurricane
100	8,800	146.6	40.0	

TO MAKE BLUE-PRINT COPIES OF TRACINGS.

The following directions, taken from *The Locomotive*, cover the whole ground. The sensitized paper can be procured at stores where artists' materials are sold, all prepared, so that the process of preparing the paper by means of chemicals can then be omitted.

The materials required are as follows:

1. A board a little larger than the tracing to be copied. The drawing-board on which the drawing and tracing are made can always be used.

2. Two or three thicknesses of flannel or other soft white cloth, which is to be smoothly tacked to the above board, to form a good smooth surface, on which to lay the sensitized paper and tracing while printing.

3. A plate of common double-thick window-glass, of good quality, slightly larger than the tracing which it is wished to copy. The function of the glass is to keep the tracing and sensitized paper closely and smoothly pressed together while printing.

4. The chemicals for sensitizing the paper. These consist simply of equal parts, by weight, of citrate of iron and ammonia, and red prussiate of potash. These can be obtained at any drug-store. The price should not be over eight or ten cents per ounce for each.

5. A stone or yellow glass bottle to keep the solution of the above chemicals in. If there is but little copying to do, an ordinary glass bottle will do, and the solution made fresh whenever it is wanted for immediate use.

6. A shallow earthen dish in which to place the solution when using it. A common dinner-plate is as good as anything for this purpose.

7. A brush, a soft paste-brush about 4 ins. wide, is the best thing we know of.

8. Plenty of cold water in which to wash the copies after they have been exposed to the sunlight. The outlet of an ordinary sink may be closed by placing a piece of paper over it with a weight on top to keep the paper down, and the sink filled with water, if the sink is large enough to lay the copy in. If it is not, it would be better to make a water-tight box about 5 or 6 ins. deep, and 6 ins. wider and longer than the drawing to be copied.

9. A good quality of white book-paper.

Dissolve the chemicals in cold water in the following proportions: 1 oz. of citrate of iron and ammonia, 1 oz. of red prussiate of potash, 8 oz. of water They may all be put into a bottle together, and shaken up. Ten minutes will suffice to dissolve them.

Lay a sheet of the paper to be sensitized on a smooth table or board; pour a little of the solution into the earthen dish or plate, and apply a good even coating of it to the paper with the brush; then tack the paper to a board by two adjacent corners, and set it in a dark place to dry; one hour is sufficient

for the drying; then place its sensitized side up, on the board on which you have smoothly tacked the white flannel cloth; lay your tracing which you wish to copy on top of it; on top of all lay the glass plate, being careful that paper and tracing are both smooth and in perfect contact with each other, and lay the whole thing out in the sunlight. Between eleven and two o'clock in the summer-time, on a clear day, from six to ten minutes will be sufficiently long to expose it; at other seasons a longer time will be required. If your location does not admit of direct sunlight, the printing may be done in the shade, or even on a cloudy day; but from one to two hours and a half will be required for exposure. A little experience will soon enable any one to judge of the proper time for exposure on different days. After exposure, place your print in the sink or trough of water before mentioned, and wash thoroughly, letting it soak from three to five minutes. Upon immersion in the water, the drawing, hardly visible before, will appear in clear white lines on a dark-blue ground. After washing, tack up against the wall, or other convenient place, bv the corners, to dry. This finishes the operation, which is very simple and thorough.

After the copy is dry, it can be written on with a common pen and a solution of common soda, which gives a white line.

HORSE-POWER, PULLEYS, GEARS, BELTING, AND SHAFTING.

Horse-power.—A horse can travel 400 yds. at a walk in 4½ minutes, at a trot in 2 minutes, and at a gallop in 1 minute; he occupies at a picket 3 ft. by 9 ft.; and his average weight equals 1,000 lbs.

An average horse carrying 225 lbs. can travel 25 miles in a day of eight hours.

A draught-horse can draw 1,600 lbs. 23 miles a day, weight of carriage included.

In a horse-mill a horse moves at the rate of 3 ft. in a second. The diameter of the track should not be less than 25 ft.

A horse-power, *in machinery,* is estimated at 33,000 lbs., raised 1 ft. in a minute; but as a horse can exert that force but six hours a day, one machinery horse-power is equivalent to that of 4 horses.

Rules to Determine the Size and Speed of Pulleys or Gears.—The driving pulley is called the *driver*, and the driven pulley the *driven*.

If the number of teeth in gears are used instead of diameter, in these calculations, number of teeth must be substituted wherever diameter occurs.

To find the diameter of driver, the diameter of the driven and its revolutions, and also revolutions of driver, being given: Multiply the diameter of driven by its revolutions, and divide the product by the revolutions of the driver; the quotient will give the diameter of the driver.

To find the diameter of driven, the revolutions of the driven, also diameter and revolutions of the driver, being given: Multiply the diameter of driver by its revolutions, and divide the product by the revolutions of the driven; the quotient will give the diameter of the driven.

To find the revolutions of the driver, the diameter and revolutions of the driven, also diameter of the driver, being given: Multiply the diameter of driven by its revolutions, and divide the product by the diameter of driver; the quotient will give the revolutions of driver.

To find the revolutions of the driven, the diameter and revolutions of the driver, also diameter of the driven, being given: Multiply the diameter of driver by its revolutions, and divide the product by the diameter of driven; the quotient will give the revolutions of driven.

Horse-power Belting will Transmit.—The ability of belting to transmit power. or to turn a wheel or "pulley," depends upon the width and thickness of the belt, the arc contact with the pulley, whether the belt is horizontal, vertical, or at an angle, and upon the velocity. The greater the velocity and the thicker the belt, the more power it will transmit. A belt running vertically or inclined will transmit less power than one running horizontally, but in figuring the horse-power capacity of belting only the velocity, width, and thickness of belt are usually considered, it being assumed that the pulleys are of proper size and located so that the belt will be nearly horizontal. Belts are commonly assumed to be of *leather*, unless otherwise designated.

The term *single belt* is used to designate a belt made of a single thickness of cowhide leather.

A double belt is made by cementing and riveting together

two thicknesses of leather. There is no standard thickness for either single or double belts.

RULES.—Many rules have been given for determining the horse-power belting will transmit.* Those most commonly used are:

For Single Belts.—Multiply the width (in inches) by the velocity in feet per minute and divide by 1,000.

For Double Belts.—Multiply the width by the velocity and divide by 700. The answer is the number of horse-power.

Some authorities give divisors of 800 and 733 for single belts, and 550 and 513 for double belts.

For the velocity of the belt multiply the number of revolutions per minute of either pulley by the circumference of that pulley.

Notes on Belting.—For continuous use a double belt is the most economical, in the long run, except on very small pulleys or for very light duty.

Triplex and quadruple belts are sometimes used for very heavy duty, but such belts are not commonly carried in stock.

Single belts should always be used with the hair side next the pulley.

The belt speed for maximum economy should be from 4000 to 4500 ft. per minute.

Idler pulleys work most satisfactorily when located on the slack side of the belt about one quarter way from the driving-pulley.

Belts are more durable and work more satisfactorily made narrow and thick, rather than wide and thin.

As belts increase in width they should also be made thicker.

For dynamo work or electric motors the ends of the belt should be fastened together by splicing and cementing, instead of lacing.

For all other cases the ends are fastened by hooks or lacing.

Belts should be cleaned and greased every five to six months.

Distance Centre to Centre of Shafts.*—In the location of shafts that are to be connected with each other by belts, care should be taken to secure a proper distance one from the other. This distance should be such as to allow of a gentle sag to the belt when in motion.

A general rule may be stated thus: Where narrow belts are to be run over small pulleys 15 ft. is a good average, the belt

* For discussion of belting, belt-dressing, care of, etc., see Kent's M. E. Pocketbook, 6th edition, pp. 876–887.

having a sag of $1\frac{1}{2}$ to 2 ins. The minimum distance between shafts is about 10 ft.

For larger belts, working on larger pulleys, a distance of 20 to 25 ft. does well, with a sag of $2\frac{1}{2}$ to 4 inches.

For main belts working on very large pulleys, the distance should be 25 to 30 ft., the belts working well with a sag of 4 to 5 ins.

If too great a distance is attempted, the belt will have an unsteady flapping motion, which will destroy both the belt and machinery.

Arrangement of Belts and Pulleys.*—If possible to avoid it, connected shafts should never be placed one directly over the other, as in such case the belt must be kept very tight to do the work. For this purpose belts should be carefully selected of well-stretched leather.

It is desirable that the angle of the belt with the floor should not exceed 45°. It is also desirable to locate the shafting and machinery so that belts should run off from each shaft in opposite directions, as this arrangement will relieve the bearings from the friction that would result when the belts all pull one way on the shaft.

If possible, machinery should be so placed that the direction of the belt motion shall be from the top of the driving to the top of the driven pulley, when the sag will increase the arc of contact.

The pulley should be a little wider than the belt required for the work, and should have a crowning face, except where the belt is to be shifted.

The motion of driving should run with and not against the laps of the belts.

Rubber belts are cheaper than leather belts and should always be used in wet places, but for ordinary use in dry places they are not as durable as leather belts.

They should always be kept free from grease or animal oils. If they slip, moisten the inside of the belt with boiled linseed oil. Some fine chalk, sprinkled on over the oil, will help the belt.

Rule for Finding the Length of Belts.—Add the diameter of the two pulleys together, multiply by $3\frac{1}{7}$, divide the product by 2, add to the quotient twice the distance

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* Kent, p. 885.

between the centre of the shafts, and the sum will be the required length.

Horse-Power Shafting will Transmit.

Diameter of Shaft in Inches.		Revolutions per Minute.						
		100	150	200	250	300	350	400
ins.	16ths.	H.P.	H.P.	H.P.	H.P.	H.P.	H.P.	H.P.
0	15	1.2	1.7	2.4	3.1	3.6	4.3	5.0
1	3	2.4	3.7	4.9	6.1	7.3	8.5	9.7
1	7	4.3	6.4	8.5	10.5	12.7	14.8	16.9
1	11	6.7	10.1	13.4	16.7	20.1	23.4	26.8
1	15	10.0	15.0	20.0	25.0	30.0	35.0	40.0
2	3	14.3	21.4	28.5	35.6	42.7	49.8	57.0
2	7	19.5	29.3	39.0	48.7	58.5	68.2	78.0
2	11	26.0	39.0	52.0	65.0	78.0	87.0	104.0
2	15	33.8	50.6	67.5	84.4	101.3	118.2	135.0
3	3	43.0	64.4	85.8	107.3	128.7	150.3	171.6
3	7	53.6	79.4	107.2	134.0	158.8	187.6	214.4
3	11	65.9	97.9	121.8	164.8	195.7	230.7	243.6
3	15	80.0	120.0	160.0	200.0	240.0	280.0	320.0
4	7	113.9	170.8	227.8	284.7	341.7	398.6	455.6
4	15	156.3	234.4	312.5	390.6	468.7	546.8	625.0

CHAIN BLOCKS.

These are portable hoisting devices which enable one man to raise a very heavy load and which will sustain the load at any point. In general, they resemble pulleys operated by chains. Since the invention of the differential pulley-block by Thos. A. Weston, about the year 1863, chain blocks have come into very general use for economical hoisting, and particularly where it is desired to hold the load at any point.

Chain blocks are of three general classes:

A. *The differential block*, which is the original and simplest and cheapest form of self-sustaining pulley.

B. *Screw- or worm-geared blocks*, of which the Yale & Towne duplex blocks are the most efficient type; and

C. *Triplex blocks*, spur-geared.

Differential and worm-geared blocks of all kinds depend upon friction to prevent the load from running down. In the triplex block a separate device is introduced which automatically holds the load safely, and yet enables it to be lowered with slight effort and at high velocity but without acceleration or danger. This is the most efficient of all chain blocks, and the most economical wherever quick work is wanted and economy in

time and labor sought. For information as to the kind of block best adapted to any particular service, the manufacturers should be consulted. The following data on the power and efficiency of chain blocks were supplied by the Yale & Towne Manufacturing Company.

Power and Efficiency of Chain Hoists.—The table below gives the work to be done by the operator at the hand-pulling chain with each size of various kinds of chain blocks in lifting the stated capacity, i.e., the amount of work or pulling required to lift this load *one foot* by stating the force exerted in pounds and the distance in feet of operating chains to be pulled. The product of these two factors determines the efficiency of the block and the ease and speed of hoisting.

Capacity in Tons.	Triplex (Spur-geared).	Duplex (Worm-geared).	Differential.
	Lbs. Ft.	Lbs. Ft.	Lbs. Ft.
½	62× 21	68× 40	122×24
1	82× 31	87× 59	216×30
1½	110× 35	94× 80	246×36
2	120× 42	115× 93	308×42
3	114× 69	132×126	557×38
4	124× 84	142×155	
5	110×126	145×195	
6	130×126	145×252	
8	135×168	160×310	
10	140×210	160×390	
12	130×126		
16	135×168		
20	140×210		

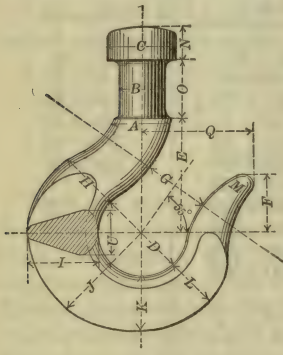
These blocks have two hand chains. The figures give the number of feet to be operated on each hand chain.

A man cannot pull more than his own weight on the operating chains, and can pull faster in proportion as the pull required is lighter. 82 lbs. is maximum pull usually required of one man, and he will do more work with less fatigue if the hand-chain pull is not over 40 lbs., because he can then pull the chain hand over hand a little more than twice as fast as he could when pulling twice as hard. When the hand-chain pull is less than 20 lbs. the speed of hoisting an equal load is diminished because the man is tired by moving his arms too rapidly, and cannot do as much work as with a heavier pull.

The best result is obtained by using a chain block having a capacity double the usual load.

The operator then works to the best advantage with average loads, and occasional heavy loads are easily handled without overstraining either the operator or the chain block, which should never be used beyond its capacity for fear of stretching the chain so that it will not work smoothly.

Proportions of Hooks.*—For economy of manufacture each size of hook is made from some regular commercial size of round iron. The basis, or initial point, in each case is, therefore, the size of iron of which the hook is to be made, which is indicated by the dimension A in the diagram. The dimension D is arbitrarily assumed. The other dimensions, as given by the formulæ, are those which, while preserving a proper bearing-face on the interior of the hook for the ropes or chains which may be passed through it, give the greatest resistance to spreading and to ultimate rupture which the amount of material in the original bar admits of. The symbol Δ is used in the formulæ to indicate the *nominal capacity* of the hook in tons of 2,000 lbs. The formulæ which determine the lines of the other parts of the hooks of the several sizes are as follows, the measurements being all expressed in inches:



$D = .5\Delta + 1.25$	$G = .75\ D$
$E = .64\Delta + 1.60$	$O = .363\Delta + \ .66$
$F = .33\Delta + \ .85$	$Q = .64\Delta + 1.60$
$H = 1.08A$	$L = 1.05A$
$I = 1.33A$	$M = \ .50A$
$J = 1\ 20A$	$N = \ .85B - .16$
$K = 1.13A$	$U = \ .866A$

Example.—To find the dimension D for a 2-ton hook. The formula is:

$$D = .5\Delta + 1.25,$$

* By Henry R. Towne, in his *Treatise on Cranes*, as the result of an extensive experimental and mathematical investigation.

and as $\Delta = 2$, the dimension D by the formula is found to be $2\frac{1}{4}$ ins.

The dimensions A are necessarily based upon the ordinary merchant sizes of round iron. The sizes which it has been found best to select are the following:

Capacity of hook	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{1}{2}$	1	$1\frac{1}{2}$	2	3	4	5	6	8	10 tons.
Dimension A.	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{7}{8}$	$3\frac{1}{4}$ inches

The formulæ which give the sections of the hook at the several points are all expressed in terms of A and can therefore be readily ascertained by reference to the foregoing scale.

EXAMPLE.—To find the dimension I in a 2-ton hook. The formula is $I = 1\ 33A$, and for a 2-ton hook $A = 1\frac{3}{8}$ in. Therefore I, in a 2-ton hook, is found to be $1\frac{13}{16}$ ins

Experiment has shown that hooks made according to the above formulæ will give way first by opening of the jaw, which, however, will not occur except with a load much in excess of the nominal capacity of the hook. This yielding of the hook when overloaded becomes a source of safety, as it constitutes a signal of danger which cannot easily be overlooked, and which must proceed to a considerable length before rupture will occur and the load be dropped. A comparison of these hooks with most of those in ordinary use will show that the latter are, as a rule, badly proportioned, and frequently dangerously weak.

THE LONGEST BRIDGES IN THE WORLD.

Forth Bridge, 9,200 ft.
Montreal Bridge, over the St. Lawrence, 8,791 ft
The Baltimore & Ohio Bridge, at Havre de Grace, 6,000 ft.
Brooklyn Bridge, over the East River, N. Y.:
- Length of river-span, 1,595 ft. 6 ins.
- Length of each land-span, 930 ft.
- Length of Brooklyn approach, 971 ft.
- Length of New York approach, 1,562 ft. 6 ins.
- Total length of bridge, 5,989 ft. Width of bridge, 86 ft.
- Number of cables, 4; diameter of each, $15\frac{3}{4}$ ins.
- Clear height of bridge in centre of river-span above high water at 90° F., 135 ft.

Williamsburg Bridge, crossing the East River at Grand St. Ferry to Brooklyn:
Extreme length, 7,250 ft.; central span, 1,600 ft.
Estimated cost $21,000,000.
Manhattan Bridge, over East River,* 2,920 ft. long in three spans. Length between terminals, 9,900 ft. Estimated cost, $13,000,000.
Blackwell's Island Bridge,* extending over Blackwell's Island, N. Y.:
Total length, 7,449 ft. Estimated cost, $18,000,000.
Wooden bridge at Columbia, Pa., 5,366 ft.
Monongahela Bridge, near Homestead, 5,300 ft.
Louisville Railroad Bridge, over the Ohio, 5,218 ft.
Volga, over the Syzran, Russia, 4,947 ft.
Moerdyck, Holland, 4,927 ft.
Dnieper, near Jékaterinoslaw, Russia, 4,213 ft.
Cincinnati Southern Railroad, over the Ohio, 3,950 ft.
Kiev, over the Dnieper, 3,607 ft.
Dauphin Bridge, over the Susquehanna, 3,590 ft.
Barrage Bridge, Delta of the Nile, 3,353 ft.
Havre de Grace Bridge, over the Susquehanna, 3,271 ft.
Kronprinz Rudolph, over the Danube at Vienna, 3,266 ft.
Dnieper, near Krementchong, Russia, 3,250 ft.
Brommel, over the Meuse, Holland, 3,060 ft.
Plattsmouth Bridge, over the Missouri, 3,000 ft.
Two bridges of Rotterdam, over the Meuse, 2,833 ft.
Quincy Bridge, over the Mississippi, 2,847 ft.
St. Louis Bridge, over the Mississippi, 2,574 ft.
Omaha Bridge, over the Missouri, 2,750 ft.
Saint-Esprit, over the Rhone, France, 2,460 ft.
Kiulmbourg, over the Rhine, Holland, 2,347 ft.
Cincinnati, over the Ohio, 2,233 ft.
Keokuk, Ia., over the Mississippi, 2,008 ft.
Chaumont Viaduct, valley of the Suize, France, 2,000 ft.
Menai, England, 1,957 ft.
Poughkeepsie Bridge, N. Y., total length 6,759 ft.

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* In process of construction.

OTHER NOTABLE BRIDGES.

The following bridges are notable either from their size or historical connection.

The Lagong Bridge, built over an arm of the China Sea, is 5 miles long, with 300 arches of stone, 70 ft. high and 70 ft. broad, and each pillar supporting a marble lion 21 ft. in length. Its cost is unknown, but much exceeds that of the Forth Bridge.

The new London Bridge is constructed of granite, from the designs of L. Rennie, and considered amongst the finest specimens of bridge architecture. It was commenced in 1824, and completed in seven years, at a cost of about $7,500,000.

The Bridge of Sighs, at Venice, over which the condemned prisoners were transported from the Judgment Hall to the place of their execution, was built in the Armada year, 1588.

The Bridge of the Holy Trinity, at Florence, consists of three beautiful elliptical arches of white marble, and stands unrivalled as a work of art. It is 322 ft. long, and was completed in 1569.

The Niagara Suspension Bridge was built in 1852–1855. It is 245 ft. above high water, 821 ft. long, and the strength is estimated at 12,000 tons.

The Rialto, at Venice, said to have been built from the designs of Michael Angelo, consists of a single marble arch, 98 ft. 6 ins. long, and was completed in 1589.

The Britannia Bridge crosses the Menai Straits, Wales, at an elevation of 103 ft. above high water. It is entirely of wrought iron, 1,511 ft. long, and was finished in 1850. Cost, $3,000,000.

The oldest bridge in England is a triangular bridge at Croyland, in Lincolnshire, which is said to have been erected about A.D. 868. It is formed of three semi-arches, whose bases stand in the circumference of a circle, equidistant from each other, and uniting at the top.

Clifton Suspension Bridge, near Bristol, has a span of 703 ft., and a height of 245 ft. above the water. The carriageway is 20 ft. wide, and footway 5½ ft. wide. Cost, $500,000.

Coalbrookdale Bridge, over the Severn, has the reputation of being the first cast-iron bridge built in England. It was erected in 1779. It consists of one arch 100 ft. wide. Total weight, 378½ tons.

DIMENSIONS AND WEIGHT OF CHURCH BELLS

MANUFACTURED BY MENEELY BELL CO., TROY, N. Y.

Bell.		Mountings.		
Weight.	Medium Tone.	Diameter.	Size of Frame, Outside.	Diameter of Wheel.
400 lbs.	D	27 in.	42×42 in.	4 ft. 4 in.
450 "	C♯	28 "	42×42 "	4 " 4 "
500 "	C	29 "	45×47 "	4 " 4 "
550 "	C	30 "	45×47 "	4 " 4 "
600 "	B	31 "	45×47 "	4 " 9 "
700 "	B	33 "	48×48 "	5 " 6 "
800 "	B♭	34 "	48×54 "	5 " 6 "
900 "	A	36 "	54×54 "	5 " 9 "
1000 "	A	37 "	54×54 "	5 " 9 "
1100 "	A	38 "	54×59 "	5 " 9 "
1200 "	A♭	39 "	56×59 "	6 " 3 "
1300 "	A♭	40 "	56×59 "	6 " 3 "
1400 "	G	41 "	60×60 "	6 " 6 "
1500 "	G	42 "	60×60 "	6 " 6 "
1600 "	G	43 "	60×60 "	6 " 6 "
1800 "	F♯	45 "	65×68 "	7 "
2000 "	F	46 "	65×68 "	7 "
2100 "	F	47 "	65×68 "	7 "
2300 "	E	49 "	70×72 "	7 " 6 "
2500 "	E	50 "	70×72 "	7 " 6 "
2800 "	E♭	51 "	74×78 "	8 "
3000 "	E♭	53 "	74×78 "	8 "
3500 "	D	56 "	74×78 "	8 " 6 "
4000 "	C♯	58 "	78×81 "	9 "
4500 "	C	61 "	78×81 "	9 "
5000 "	C	63 "	84×84 "	9 "
5500 "	B	65 "	84×84 "	9 "
6000 "	B♭	67 "	84×84 "	9 " 6 "
6500 "	B♭	68 "	90×90 "	9 " 6 "

The actual weights usually exceed the patterns, noted above, from two to three per cent.

MENEELY SCHOOL BELLS.

Bell.		Mountings.
Weight.	Diameter.	Size of Frame, Outside.
100 pounds	17 inches	2 feet 6 inches by 2 feet 8 inches
125 "	18½ "	2 " 6 " " 2 " 8 "
150 "	19½ "	2 " 6 " " 2 " 8 "
175 "	20½ "	2 " 8 " " 3 " 0 "
200 "	21½ "	2 " 8 " " 3 " 0 "
225 "	22 "	2 " 8 " " 3 " 0 "
250 "	23 "	3 " 0 " " 3 " 2 "
275 "	24 "	3 " 0 " " 3 " 2 "
300 "	24½ "	3 " 0 " " 3 " 4 "
325 "	25 "	3 " 0 " " 3 " 4 "
350 "	26 "	3 " 0 " " 3 " 4 "
375 "	26½ "	3 " 0 " " 3 " 4 "

SIZE OF ROPE FOR BELLS.

For bells of less than 500 pounds.	½	inch diameter
" " " 500 to 800 pounds.	⅝	" "
" " " 800 to 1,800 pounds.	¾	" "
" " above 1,800 pounds.	⅞ to 1	" "

THE LARGEST RINGING BELLS IN THE WORLD.*

Names and Location of Bells.	Date Cast.	Actual Vibration.	Key-note.	Diameter, Inches.	Sound-bow. Inches.	Sound-bow. Stroke.	Weight, Pounds.
Moscow, Tzar Kolokol.......	1733	74	D	272	23	0.84	443,772
Burmah, Mengoon..........	....	94	F♯	203?	16?	0.80	201,600
Moscow, St. Ivan's.........	1819	105	G♯	185	14.75	0.80	127,350
Pekin, Great Bell...........	....	....	....	156	.....	.....	120,000
Burmah, Maha Ganda.......	....	125	B	155	12.5	0.80	95,000
Nishni Novgorod...........	....	125	B	151	12	0.80	69,664
Moscow, Church of Redeemer	1879	141	C♯	136.3?	10.6	0.80	60,736
Nankin, China.............	....	....	....	112	......	.....	45,000
London, St. Paul's.........	1881	157	E♭	114.25	8.75	0.76	42,000
Olmutz, Bohemia...........	....	157	E♭	121	9.125	0.75	40,320
Vienna, Austria............	1711	157	E♭	118	9.5	0.80	40,200
Westminster, London.......	1856	166	E	113.5	9.375	0.83	35,620
Erfurt, Saxony.............	1487	176	F	103.6	9.75	0.75	30,800
Notre Dame, Paris..........	1680	166	E	103	7.5	0.73	28.670
Montreal, Canada...........	1847	176	F	103	7.8	0.76	28,560
York, England..............	1845	187	F♯	100	8	0.80	24,080
St. Peter's, Rome..........	1786	187	F♯	97.25	7.5	0.77	18,000
Great Tom, Oxford.........	1680	210	G♯	84	6.125	0.73	17,024
Cologne, Germany..........	1477	198	G	95	7.2	0.76	16,016
Brussels, Belgium	....	210	G♯	95.81	7.75	0.71	15,848
State-house, Philadelphia....	1875	198	G	88	6.375	0.73	13,000
Lincoln, England...........	1834	210	G♯	82.85	6	0.73	12,096
St. Paul's, London.........	1716	222	A	81	6.08	0.75	11,500
Exeter, England............	1675	210	G♯	76	5	0.66	10,080
Old Lincoln, England.......	1610	249	B	75.5	5.94	0.78	9,856
Westminster, London.......	1857	249	B	72	5.75	0.79	8,960

* John W. Nystrom, in the Journal of the Franklin Institute.

SYMBOLS FOR THE APOSTLES AND SAINTS.

From the constant occurrence of symbols in the edifices of the Middle Ages and many of the cathedrals of the present day, the following list of symbols, as commonly attached to the apostles and saints, may be found useful:

Holy Apostles.

St. Peter.—Bears a key, or two keys with different wards.

St. Andrew.—Leans on a cross so called from him; called by heralds the saltire.

St. John the Evangelist.—With a chalice, in which is a winged serpent. When this symbol is used, the eagle, another symbol of him, is never given.

St. Bartholomew.—With a flaying-knife.

St. James the Less.—A fuller's staff bearing a small square banner.

St. James the Greater.—A pilgrim's staff, hat, and escalop-shell.

St. Thomas.—An arrow, or with a long staff.

St. Simon.—A long saw.

St. Jude.—A club.

St. Matthias.—A hatchet.

St. Philip.—Leans on a spear or has a long cross in the shape of a T.

St. Matthew.—A knife or dagger.

St. Mark.—A winged lion.

St. Luke.—A bull.

St. John.—An eagle.

St. Paul.—An elevated sword, or two swords in saltire.

St. John the Baptist.—An Agnus Dei.

St. Stephen.—With stones in his lap.

Saints.

St. Agnes.—A lamb at her feet.

St. Cecilia.—With an organ.

St. Clement.—With an anchor.

St. David.—Preaching on a hill.

St. Denis.—With his head in his hands.

St. George.—With the dragon.

St. Nicholas.—With three naked children in a tub, in the end whereof rests his pastoral staff.

St. Vincent.—On the rack.

HEIGHTS OF COLUMNS, TOWERS, DOMES, SPIRES, ETC.

COLUMNS.

Name.	Place.	Feet
Alexander.	St. Petersburg	175
Bunker Hill.	Charlestown, Mass.	$221\frac{1}{6}$
Chimney (St. Rollox).	Glasgow.	$455\frac{1}{2}$
Chimney (Musprat's).	Liverpool.	406
City.	London.	202
July.	Paris.	157
Napoleon.	Paris.	132
Nelson's.	Dublin.	134
Nelson's.	London.	171
Place Vendôme	Paris.	136
Pompey's Pillar.	Egypt.	114
Trajan.	Rome.	145
Washington.	Washington.	555
York.	London.	138

TOWERS AND DOMES.*

Name.	Place.	Feet.
Eiffel Tower.	Paris.	985
Tower.	Babel.	680
Tower.	Baalbec.	500
Cathedral (spire).	Cologne.	$516\frac{3}{4}$
Cathedral.	Rouen.	$491\frac{2}{3}$
Cathedral (spire).	Antwerp.	476
St. Nicholas.	Hamburg.	473
Cathedral.	Anvers.	472
St. Peter's (cupola)	Rome.	$469\frac{1}{6}$
Cathedral.	Cremona.	392
Cathedral.	Escurial.	200
Cathedral.	Florence.	384
Cathedral.	Milan.	438
Cathedral.	St. Petersburg.	363
Capitol (dome).	Washington.	$287\frac{1}{2}$
Leaning Tower.	Pisa.	188
Porcelain.	China.	200
St. Paul's.	London.	366
St. Mark's.	Venice.	328
City Hall.	Philadelphia.	$537\frac{1}{3}$

* See also next page.

HEIGHT OF SPIRES.

Name.	Place.	Feet.
Cathedral	Strasburg	$465\frac{1}{6}$
Cathedral, new	New York	325
Grace Church	New York	216
Cathedral	Salisbury	450
St. John's	New York	210
St. Paul's	New York	200
St. Mary's	Lübeck	404
St. Peter's	Rome	391
St. Stephen's	Vienna	465
Trinity Church	New York	286
Balustrade of Notre Dame	Paris	216
Hôtel des Invalides	Paris	344
Pyramid of Cheops	Egypt	520
Pyramid of Sakara	Egypt	356

LIST OF THE PRINCIPAL DOMES IN THE WORLD.

Their diameter and height from the ground.
(Gwilt's Encyclopædia.)

Place.	Diam., Feet.	Height, Feet.
Pantheon, at Rome	142	143
Duomo, or Sta. Maria del Fiore, at Florence	139	310
St. Peter's, at Rome	139	330
Sta. Sophia, at Constantinople	115	201
Baths of Caracalla (ancient)	112	116
St. Paul's, London	112	215
Mosque of Achmet	92	120
Chapel of the Medici	91	199
Baptistery, at Florence	86	110
Church of the Invalids, at Paris	80	173
Minerva Medica, at Rome	78	97
Madonna della Salute, Venice	70	133
St. Généviève, at Paris (Panthéon)	67	190
Duomo, at Sienna	57	148
Duomo, at Milan	57	254
St. Vitali's, at Ravenna	55	94
Val de Grace, at Paris	55	133
San Marco, Venice	44	
United States Capitol, Washington	$124\frac{3}{4}$	$307\frac{1}{2}$

See page 1533.

PRINCIPAL DIMENSIONS OF THE ENGLISH CATHEDRALS.

(Gwilt.)

Cathedral.	Total Internal Length, in Feet.	Naves and Aisles.			Choirs.			Transepts.	Spires and Towers.	
		Length, in Feet.	Breadth, in Feet.	Height, in Feet.	Length, in Feet.	Breadth, in Feet.	Height, in Feet.	Breadth, in Feet.		Height, in Feet.
Winchester	545	247	86	78	138	—	73	186		
Ely	517	327	73	70	101	73	70	178	Tower	210
Canterbury	514	214	70	80	150	74	80	154	Tower	235
Old St. Paul's	500	335	91	102	165	42	88	248	Spire	534
York	498	264	109	99	131	—	99	222	Tower	234
Lincoln	498	—	83	83	—	—	—	227	Tower	260
Westminster	489	130	96	101	152	—	151	189		
Peterborough	480	231	78	78	138	—	78	203	Louvre	150
Salisbury	452	246	76	84	140	—	84	210	Spire	387
Durham	420	—	—	—	117	33	71	176	Tower	214
Gloucester	420	174	84	67	140	—	86	144	Tower	225
Lichfield	411	213	67	—	110	—	67	—	Spire, 258 W.	183
Norwich	411	230	71	—	165	—	—	191	Spire	317
Worcester	410	212	78	—	126	—	74	130	Tower	196
Chichester	401	205	91	61	100	—	—	131	Spire	267
Exeter	390	173	74	69	131	—	69	140	Tower	130
Wells	371	191	67	67	106	—	67	135	Tower	160
Hereford (ancient)	370	144	68	68	105	—	64	140		
Chester	348	—	73	73	—	—	—	—	Tower	127
Rochester	306	150	65	—	156	—	—	122	Spire	156
Carlisle	213	—	71	71	137	71	—	—		
Bath	210	136	72	78	—	—	—	126	Tower	162
Bristol	175	100	75	73	100	—	—	128	Tower	127
Oxford	154	74	54	41	80	—	37½	102	Spire	184

DIMENSIONS OF THE VARIOUS OBELISKS EXISTING AT THE PRESENT TIME.

(Gwilt's Encyclopædia.)

Situation.	Height, in English Feet.	Thickness, in English Ft. At Top.	Thickness, in English Ft. Below.
Two large obelisks mentioned by Diodorus Siculus	158.2	7.9	11.8
Two obelisks of Nuncoreus, son of Sesostris, according to Herodotus, Diodorus Siculus, and Pliny	121.8	6.6	10.5
Obelisk of Rhameses, removed to Rome by Constantius	118.4	6.2	10.2
Two obelisks, attributed by Pliny to Smerres and Eraphius	106.0	5.9	9.8
Obelisks of Nectanabis, erected near the Tomb of Arsinöe by Ptolemy Philadelphus	105.5	5.3	9.2
Obelisk of Constantius, restored and erected in front of S. Giovanni Laterano, at Rome	105.5	6.2	9.6
Part of one of the obelisks of the son of Sesostris, in the centre of the piazza in front of St. Peter's	82.4	5.8	9.4
Two at Luxor	79.1	5.3	8.0
Obelisk of Augustus, from the Circus Maximus, now in the Piazza del Popola at Rome	78.2	4.5	7.4
Two in the ruins at Thebes, still remaining	72.8	5.0	7.5
Obelisk of Augustus, raised by Pius VI. in the Piazza di Monte Citorio	71.9	4.9	7.9
Two obelisks: one at Alexandria, vulgarly called Cleopatra's Needle, and the other at Heliopolis	67.1	5.1	8.1
Obelisk by Pliny, attributed to Sothis	63.3	4.5	5.1
Two obelisks in the ruins at Thebes	63.3	4.5	5.1
Great obelisk at Constantinople	59.7	4.5	7.2
Obelisk in the Piazza Navona, removed from the Circus of Caracalla	54.9	2.9	4.5
Obelisk at Arles	50.1	4.5	7.4
Obelisk from the Mausoleum of Augustus, now in front of the Church of Sta. Maria Maggiore, at Rome	48.3	2.9	4.3
Obelisk in the Gardens of Sallust, according to Mercati	48.3	2.9	4.3
Obelisk at Bijije, in Egypt	42.9	2.6	4.2
Small obelisk at Constantinople, according to Gyllius	34.2	3.9	5.9
The Barberini Obelisk	30.0	2.2	3.9
Obelisk of the Villa Mattei	26.4	2.2	2.7
Obelisk in the Piazza della Rotunda	20.1	2.1	2.4
Obelisk in the Piazza di Minerva	17.6	2.0	2.6
Obelisk of the Villa Medici	16.1	1.9	2.4

DIMENSIONS OF SOME WELL-KNOWN EUROPEAN BUILDINGS.*

The body of Milan Cathedral, from the great doorway to the end of the apse, measures 148 metres and 10 centimetres, with a breadth of 57 metres. The total length of the transepts with the chapels is 87 metres. The nave is 47 metres high by 19 in width, and the total height, from the centre to the feet of the statue of the Virgin which crowns the central tower, is 108.5 metres.

The Cathedral of York, burned in 1828, and which had already been rebuilt in 1705, has a length of 142 English feet, a breadth of 105 feet at the western extremity, and 109 feet at the opposite end. The total height of the nave is 99 feet; the ceiling of the central tower is 213 feet from the ground. A window which opens at the extremity of the gallery, and which is entirely filled with stained glass, is 65 English feet in height by 32 in width.

The Cathedral of Cordova, built in the year 792 by King Abderame, is 134 feet long and 387 wide. This church contains nine naves formed by 1,018 columns, the smallest of which are 7 feet and the largest 11 feet and 3 inches high.

The Escurial, begun in 1557, to which was given the form of a gridiron, in honor of St. Lawrence, is 51 feet in height and 637 feet in length.

In the Alhambra at Granada, an ancient Moorish fortress, the Lion Court is 100 feet square.

The Church of St. Denis, near Paris, is 335 feet long by 90 feet high. It was built in 1152 by Suger.

The famous Column of the Grand Army on the Place Vendôme, Paris, is 136 feet high.

The Church of St. Généviève, at Paris, to-day transformed into the Panthéon, is one of the most remarkable structures by reason of the vastness of its proportions. The diameter of the dome is 68 feet. The 32 columns which surround it are 34 feet in height, and the highest point of the edifice is 237 feet from the sidewalk.

The Cathedral at Rheims, which Stendhal considers one of the most beautiful churches in France, was built in 840, and measures 430 feet in length by 110 in height.

The Cathedral at Strasburg, which is perhaps the only purely Gothic monument on the Continent of Europe, was finished in 1275. The first stone was laid in 1015. The tower, finished in

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* Taken from an article on Milan Cathedral, published in the *American Architect*, August 25, 1888.

1439, is, without contradiction, the highest bit of masonry which exists in Europe. Its height is 426 feet; width of nave, 43 feet; length, 145 feet, inside measurements.

The tower of St. Etienne at Vienna is 414 feet high, 4 feet less than that at Strasburg.

The tower of St. Michael at Hamburg is 390 feet.

The famous tower of Pisa measures 193 feet, but it leans toward the south about 12 feet, which gives it a mean inclination of 6 feet in the hundred.

St. Sophia, at Constantinople, measures 270 feet in length by 240 feet in width, from north to south. The height of the dome above the level of the ground is only 165 feet.

The towers of Notre Dame, at Paris, measure 240 feet in height. The total length of this church is 409 feet. Its interior width at the crossing is 150 feet; the width of the nave is 40 feet.

The Church of St. Paul, at London, is 500 feet in length by 169 feet in width. The height of the dome is 319 feet.

St. Peter's, at Rome; total length, including the portico and thickness of the walls, is 660 feet. The foundation walls are 21 feet and 7 inches thick. The walls of the peristyle are 8 feet and 9 inches thick, and the peristyle is 39 feet and 3 inches in width. The interior length of the crossing of St. Peter's is 98 feet. The interior width of the nave, without the aisles and chapels, is 82 feet. The total height from the floor to the summit of the cross which surmounts the dome is 408 feet. The height of the dome under the key-stone is 249 feet. The interior height of the façade is 259 feet.

DIMENSIONS OF THE GRAND OPERA-HOUSE, PARIS.

Superficial area, 37,317 square feet, and cubical contents, 428,660 metres.

The width of the façade is 230 feet.

Greatest width of building, 408 feet.

Height above the ground level, 184 feet.

From foundation to summit, 266 feet.

No less than fifteen eminent painters, fifty-six eminent sculptors, besides nineteen sculptors of ornament, were engaged on the external and internal decorations.

M. Garnier, the architect, gave his entire and unremitting attention to it, and, with the aid of his assistants, produced more than 30,000 drawings. The building was in course of construction for thirteen years.

HEIGHT OF SOME OF THE TALLEST BUILDINGS IN THE UNITED STATES.

BUILDINGS IN NEW YORK CITY.

Building	Stories		Height above Sidewalk	
Ivins Syndicate (Park Row) Building[1]	29 stories		386	ft.
New Times Building[16]	22	"	375*	"
Manhattan Life Building[2]	18	" and tower	348	"
International Bank Bld'g.[3] 60 Wall St.	26	" Pine St. wing	345	"
Wall St. Exchange[3]	25	"	340	"
St. Paul Building[5]	26	"	313	"
American Surety Building[6]	21	"	312	"
Pulitzer (World) Building[5]	16	" and dome	309	"
Hanover National Bank[10]	22	"		"
American Tract Society Building[1]	21	"	306	"
Empire Building[2]	20	"	304	"
Commercial Cable Building[7]	20	"	304	"
Whitehall (Battery Place) Building[17]	20	"		"
Forty-two Broadway Building[19]	20	"		"
Madison Square Garden[8]	to top of tower		300	"
Gillender Building[9]	19 stories and tower		300	"
Fuller (Flat Iron) Building[21]	21	"	285	"
Trinity Church spire			284	"
Standard Oil Building[2] (remodelled)	19 stories		280	"
Broad Exchange Building[3]	20	"	276	"
Bank of Commerce Building[10]	19	"	264	"
Broadway-Maiden Lane Building[3]	18	"	234	"
Broadway Chambers[14]	18	"		"
Home Life Insurance Building[11]	16	" and tower	257	"
Washington Building[12]	13	" " "	250	"
New York Life Building[8]	12	" " "	244	"
S. L. Mitchell Estate Building	15	"	230	"
Mutual Life Building[4]	14	"	230	"
Manhattan Hotel	16	"	225	"
Produce Exchange Building[5]	9	" and tower	225	"
Queens Insurance Co. Building[7]	17	"		"
Bowling Green Building[13]	16	"	224	"
St. James Building[6]	16	"		"
American Exchange Bank[3]	16	"		"
New Netherlands Hotel	16	"	220	"
Blair Building[15]	16	"		"
Bank of the Metropolis[6]	16	"		"
Beaver Building[3]	15	"		"
Dun Building[7]	15	"	223	"
Central Bank Building[20]	15	"	219	"
Hudson Building	16	"	218	"
Lords Court Building[18]	15	"	214	"
Johnston Building	15	"	212	"
Syndicate Building	15	"	207	"

* 430 ft. above footings.

		Height above Sidewalk.	
Continental Ins. Co. Building	14 stories	215	ft.
Union Trust Building [5]	to top of tower	194	"
Postal Telegraph Building [7]	13 stories	192	"
Havemeyer Building [5]	14 "	192	"
Mutual Reserve Building	13 "	184	"
Times Building (old) [5]		183	"
Silk Exchange Building	13 stories	180	"

ARCHITECTS.—[1] R. H. Robinson. [2] Kimball & Thompson. [3] Clinton & Russell. [4] C. W. Clinton. [5] Geo. B. Post. [6] Bruce Price. [7] Geo. Edward Harding & Gooch. [8] McKim, Mead & White. [9] Berg & Clark. [10] James B. Baker. [11] N. Le Brun & Sons. [12] E. H. Kendall. [13] Audsley Bros. [14] Cass Gilbert. [15] Carrere & Hastings. [16] Cyrus L. W. Eidlitz. [17] Henry J. Hardenbergh. [18] John T. Williams. [19] Henry Ives Cobb. [20] Wm. H. Birkmire. [21] D. H. Burnham & Co.

CHICAGO.

			Height above Sidewalk.	
Masonic Temple [22]	20 stories	Roof line	278	ft.
To top of skylight			303	"
Auditorium [23]	17 stories and tower		265	"
Fischer Building [21]	18 "	" attic	235	"
Old Colony Building [24]	17 "		213	"
Schiller Theatre [23]	17 "			"
Katahdin & Wachusett Building	17 "		203½	"
Unity Building	17 "		210	"
Railway Exchange Building [21]	17 "			"
Marquette Building [24]	16 "		207	"
Monadnock Building	16 "		215	"
Ashland Block	16 "		200⅔	"
The New Great Northern Building [21]	16 "		200	"
Manhattan Building [30]	16 "		197	"
Reliance Building [21]	14 "		200	"
Security Building	14 "		200	"
Title & Trust Building	16 "		198	"
Woman's Temple [22]	13 "	Ridge	198	"
Champlain Building [24]	15 "		189	"

BOSTON.

Ames Building [25]	Top of cornice	186	ft.
Chamber of Commerce [25]	Top of tower	172½	"

BUFFALO, N. Y.

Guaranty Building [23] .... 13 stories ....

CINCINNATI.

Ingalls Building [27] .... 15 stories (concrete-steel construct'n)

PHILADELPHIA.

City Hall	To top of tower	537	ft.
Land & Title Building	22 stories	317	"

PITTSBURG.

Allegheny County Court House [26]	To top of finial	319	ft.
Farmers' Bank Building [28]	25 stories		"

SAN FRANCISCO.

Spreckels Building [29]	16 stories and tower	215	ft.

MISCELLANEOUS.

U. S. Capitol, Washington	Top of dome	307½	ft.
State Capitol, Hartford, Conn	Top of figure on dome	256	"

ARCHITECTS.—[21] D. H. Burnham & Co. [22] Burnham & Root. [23] Adler & Sullivan. [24] Holabird & Roche. [25] Shepley, Rutan & Coolidge. [26] H. H. Richardson and Shepley Rutan & Coolidge. [27] Elzner & Anderson. [28] Alden & Harlow. [29] Reid Bros. [30] Jenney & Mundie.

DESCRIPTION OF NOTABLE AMERICAN BUILDINGS.

THE UNITED STATES CAPITOL.

[From "King's Hand-book of Washington."]

The site of the building is 89½ ft. above ordinary low tide in the Potomac. Entire length of building, 751 ft.; greatest depth (breadth of wings), 324 ft.; area covered by building, 3½ acres. The central building is 352 ft. long; corridors, 44 ft. long; wings, 143 ft. front, 239 ft. deep, exclusive of porticos and steps. Central building is freestone from quarries about 40 miles below Washington. This is painted white.

The wings are of white marble from Lee, Mass. Appropriations made by Congress from 1800 to date for the erection and remodelling of the Capitol amount to $15,000,000.

Dome designed by T. U. Walter, to replace a smaller one removed in 1856. Exterior height crest of statue above base-line, 307½ ft.; top of lantern above balustrade of building, 218 ft.; height of Statue of Freedom on the apex, 19½ ft.; diameter of dome, 135½ ft.

The dome rests on an octagonal base 93 ft. above the basement floor, and as it leaves the top line of the building consists of a peristyle, 124 ft. in diameter, of 36 iron-fluted columns 27 ft. high and weighing 6 tons each.

The lantern is 15 ft. in diameter and 50 ft. high.

The weight of iron in the superstructure of the dome is 8,009,-200 lbs. This rests on a substructure of masonry and 40 interior massive stone columns supporting heavy groined arches, upon which also rests the pavement of the Rotunda.

Height from floor of Rotunda to canopy, 180 ft.; diameter of Rotunda, 96 ft.

The canopy consists of an inner shell of iron ribs and lathing, laid with plaster suitable for frescoing. It is 65½ ft. in diameter, and 21 ft. vertical height.

Supreme Court Room.—Seventy-five ft. long, 45 ft. wide, and 45 ft. high.

Hall of Representatives.—Length, 139 ft.; width, 93 ft.; height, 36 ft.; floor, 115 ft. by 67 ft. Galleries will seat about 2,500 persons.

The ceiling of the hall is of cast iron, panelled, painted, and gilded, and highly enriched with gilt mouldings. The panels are filled with glass, with stained centre-pieces representing the arms of the States. Above the ceiling is the illumination-loft, with 1,500 gas-jets, for lighting the hall for night sessions.

Senate Chamber.—Length, 113¼ ft.; width, 80¾ ft.; height, 39 ft.

Floor is 83 ft. long, 51 ft. wide. Galleries seat 1,200 persons. The ceiling is of iron with glass panels, lighted same as Representatives' Hall.

The Congressional Library.—In response to an invitation for competitive plans, 28 designs were submitted, from among which that of Messrs. Smithmeyer & Pelz of Washington was selected as the best, and they were entrusted with the work. Mr. Smithmeyer was early retired, and in 1892 Mr. Pelz was also retired, and after that Mr. Edward P. Casey took up the work. In many respects it is one of the most notable buildings in this country.

The dimensions of the ground plan are: Length of frontage, 470 ft., depth, 340 ft. The reading-room is 100 ft. inscribed diameter. The niches are 18 ft. additional, making the open space between opposite walls 136 ft. The stair hall is 48′× 80′, but with adjoining corridors and between walls, is 94′× 136′. The longest rooms, the north and south wings, are 210′× 35′. There are three book repositories, with a total capacity of about 2,000,000 volumes.*

* The Architects' and Builders' Magazine, July, 1900.

Treasury Building.—Dimensions: 468 ft. north to south, 264 ft. east to west; inclusive of porticos and steps, 582 ft. by 300 ft. Cost, $6,000,000.

Architects—Robert Mills, T. U. Walter, Young, Rogers, and A. B. Mullett.

State, War, and Navy Building.—A. B. Mullett, architect. Extreme dimensions north to south, 567 ft.; east to west, 342 ft.; exclusive of projection, 471 ft. north to south, and 253 ft. east to west. Cost, $5,000,000.

New City Hall, Philadelphia; John McArthur, Jun., architect.

Dimensions of Building

From north to south	486 ft. 6 ins.
" east to west	470 ft.
Area	4½ acres
Number of rooms in building	520
Total amount of floor-room	14½ acres
Height of main tower	537 ft. 4 ins.
Width at base	90 "
Centre of clock-face above pavement	361 "
Diameter of clock-face	20 "

State Capitol, Hartford, Conn.; R. M. Upjohn, architect, New York City.

Exterior is of marble; building is of fire-proof construction, with brick and iron floors.

Length	296 ft.
Depth	199 "
Height to top of roof	99 "
Height to top of figure on dome	256 "
Senate chamber	50 " × 40 ft., 35 ft. high
Representatives' hall	84 " × 56 " 48 " high
Supreme Court room	50 " × 31 " 35 " high

Cost of building, $2,500,000.00.

The Washington Monument, at Washington, D.C., is 555 ft. 5 ins. high, and has a base of 55 ft., with an entasis of 1 ft. in every 34 in height. The monument is faced with white marble and backed with blue granite to the height of 452 ft.; above that the walls are entirely of marble. The average settlement of the structure at each corner is 1.7 ins. The monu-

ment is a simple plain obelisk with no embellishments whatever.

The weight of the monument is 80,470 tons, or 3.6 tons per square foot; the area covered by the foundation being 22,400 sq. ft.

The corner-stone of the monument was laid July 4, 1848, and the cap-stone was set Dec. 6, 1884.

The Madison Square Garden, New York City.—Messrs. McKim, Mead & White, architects. This building covers the block bounded by East Twenty-seventh Street, Fourth Avenue, Twenty-sixth Street, and Madison Avenue. It was opened to the public June 16, 1890, and cost $3,000,000.

It combines an immense amphitheatre, a restaurant (80′ × 90′), a ball-room, a concert hall, an open-air roof garden (80′ × 200′), and a theatre.

The amphitheatre is an enormous room, 310′ × 194′ and 80′ high, with an arena containing 30,000 sq. ft. The room is semicircular at each end, and is provided with permanent seats for 7,800 people, with sufficient standing space left to give room for a total of 15,000 persons. This vast arena, covered by the immense roof without central support, is entirely open and free from side to side and from end to end. For summer performances the roof can be opened by machinery.

The theatre has a seating capacity of about 1,200, with standing room for 400 more.

The open-air garden extends over the roof along the Madison Avenue front. It will hold from 3,000 to 5,000 people.

The building is surmounted by an immense tower 300 ft. high.

Auditorium Building, Chicago, Ill.; Adler & Sullivan, architects.

This building was built during the years 1887–89 and includes:

1. *The Auditorium.*—Permanent seating capacity, over 4,000; for conventions, etc. (for which the stage will be utilized), about 8,000. Contains the most complete and costly stage and organ in the world.

2. *Recital Hall.*—Seats over 500.

3. *Business Portion* consists of stores and 136 offices, part of which are in the tower.

4. *Tower Observatory*, to which the public are admitted.

Above four departments of the building are managed by Chicago Auditorium Association.

5. *Auditorium Hotel* has 400 guest rooms. The grand dining-

room (175 feet long) and the kitchen are on the top floor. The magnificent banquet hall is built of steel, on trusses, spanning 120 feet over the Auditorium.

Area covered by building, about one and one-half acres.

Cost of building, $3,200,000.

ARCHITECTS OF NOTED PUBLIC AND SEMI-PUBLIC BUILDINGS IN THE UNITED STATES.

Buildings Arranged according to Location.

GOVERNMENT BUILDINGS IN WASHINGTON, D. C.

	Architects.
United States Capitol	Messrs. Hallet, Hadfield, Hoban, Latrobe, Bulfinch, Walter, and Clark.
National Museum	Cluss & Schulye.
State, War and Navy Building	A. B. Mullett.
Treasury Building	Robert Mills, T. U. Walter, Young, Rogers, and A. B. Mullett.
The Congressional Library	Smithmeyer & Pelz, Edward P. Casey.*

United States Post Offices and Court-houses:

Location.	
Baltimore, Md.	James G. Hill.
Boston, Mass.	A. B. Mullett.
Chicago, Ill. (old)	A. B. Mullett.
Chicago, Ill. (new)	Henry Ives Cobb.
Cincinnati, O.	A. B. Mullett.
Detroit, Mich.	M. E. Bell.
Kansas City, Mo.	James G. Hill.
New York, N. Y.	A. B. Mullett.
St. Louis, Mo.	A. B. Mullett.

Other Government Buildings.

Immigrant Station, Ellis Island, N. Y. Harbor	Boring & Tilton.
New Naval Academy, Annapolis, Md.	Ernest Flagg.

* See p. 1534.

STATE CAPITOLS.

Capitol of:

	Architects.
Colorado, at Denver	E. E. Meyers & Son.
Connecticut, at Hartford	R. M. Upjohn.
Illinois, at Springfield	A. H. Piquenard.
Indiana, at Indianapolis	Edwin May.
Iowa, at Des Moines	A. H. Piquenard.
Georgia, at Atlanta	W. J. Edbrook & F. P. Burnham.
Louisiana, at Baton Rouge	W. A. Freret.
Maine, at Augusta	Charles Bulfinch.
Massachusetts, at Boston	Charles Bulfinch; Brigham & Spofford.
Michigan, at Lansing	E. E. Meyers.
Minnesota, at St. Paul,	Cass Gilbert.

Capitol of:

New York, at Albany	Messrs. Fuller, Eidlitz, and H. H. Richardson.
Ohio, at Columbus	Henry & Wm. Walter.
Rhode Island, at Newport	James Munday.
Tennessee, at Nashville	John Strickland.
Texas, at Austin	E. E. Meyers & Son.
Virginia, at Richmond	Thomas Jefferson.

COUNTY BUILDINGS.

Court-house, Baltimore	Wyatt & Nolting.
Suffolk County Court-house, Boston, Mass.	Geo. A. Clough.
Cook County Court-house, Chicago, Ill.	J. J. Egan.
Arapahoe County Court-house, Denver, Col.	E. E. Meyers & Son; F. Eberley.
Jefferson Market Court-house, New York	F. C. Withers.
The Appellate Division Court-house, New York	James Brown Lord.
Allegheny County Court-house and Jail, Pittsburgh, Pa.	H. H. Richardson.
Court-house, Providence, R. I.	Stone & Carpenter.

CITY AND TOWN HALLS.

City Hall:
- Albany, N. Y. . . . . H. H. Richardson.
- Boston, Mass. . . . . Gilman & Bryant.
- Detroit, Mich. . . . . James Anderson.
- New York, N. Y. (1803–12) . . John McComb.
- (New) Philadelphia, Pa. . . . . . John McArthur, Jr.
- Worcester, Mass. . . . . Peabody & Stearns.

Town Hall, North Easton, Mass. . H. H. Richardson.

LIBRARIES.

Name and Location.	Architect.
Public Library, Boston, Mass.	McKim, Mead & White.
Public Library, Chicago, Ill.	Shepley, Rutan & Coolidge.
Newberry Library, Chicago.	Henry Ives Cobb.
Lenox Library, New York.	R. M. Hunt.
Free Circulating Library, N. Y. Branch No. 1 *	James Brown Lord.
Chatham Sq. Branch N. Y. Pub. Library *.	McKim, Mead & White.
Blackstone Memorial Library, Branford, Conn.	S. S. Beman.
Public Library, Erie, Pa	Alden & Harlow.
Public Library, Mankato, Minn.*. .	Jardine, Kent & Jardine.
Public Library, Milton, Mass.	Shepley, Rutan & Coolidge.
Public Library, Tacoma, Wash. . . .	Jardine, Kent & Jardine.
Public Library, Milwaukee, Wis. . .	Ferry & Class.
Public Library, Newark, N. J.	Rankin & Kellogg.
Public Library, Schenectady, N. Y.*	Penn Varney.
Carnegie Library, Syracuse, N. Y. .	Jas. A. Randall.
Carnegie Library, Paducah, Ky. . . .	A. L. Lassiter.
Carnegie Library, East Orange, N.J.	Jardine, Kent & Jardine.
Carnegie Library, Sandusky, O. . . .	D'Oench & Yost

ART INSTITUTES AND MUSEUMS.

Museum of Fine Arts, Boston.	Sturgis & Brigham.
Academy of Fine Arts, Chicago. . .	Burnham & Root.
Art Institute, Chicago	Shepley, Rutan & Coolidge.
Art Museum, Detroit.	James Balfour.
Museum of Fine Arts, St. Louis . . .	Peabody & Stearns.

* Carnegie Libraries.

LIST OF NOTED ARCHITECTS.

(Gwilt.)

Before Christ.

Name of Architect.	Century.	Principal Works.
Theodorus, of Samos.	7th	Labyrinth at Lemnos, some buildings at Sparta, and the Temple of Jupiter at Samos.
Ictinus, of Athens.	6th	Parthenon at Athens, Temple of Ceres and Prosperpine at Eleusis, Temple of Apollo Epicurius in Arcadia.
Callicrates, of Athens.	6th	Assisted Ictinus in the erection of the Parthenon.
Mnesicles, of Athens.	6th	Propylæa of the Parthenon.
Dinocrates, of Macedonia.	4th	Rebuilt the Temple of Diana at Ephesus, engaged on works at Alexandria, was the author of the proposition to transform Mount Athos into a colossal figure.
Andronicus, of Athens.	4th	Tower of the Winds at Athens.
Callimachus, of Corinth.	4th	Reputed inventor of the Corinthian order.
Sostratus, of Cnidus.	4th	The Pharos of Alexandria.
Cossutius, of Rome.	2d	Design for the Temple of Jupiter Olympus at Athens.
Hermodorus, of Salamis.	2d	Temple of Jupitor Stator in the Forum at Rome, Temple of Mars in the Circus Flaminius.
Fussitius, of Rome.	1st	Several buildings at Rome; the first Roman who wrote on architecture.

After Christ.

Name of Architect.	Century.	Principal Works.
Virtruvius Pollio, of Fano.	1st	Basilica Justitiæ at Fano; a great writer on architecture.
Metrodorus, of Persia.	4th	Many buildings in India and some at Constantinople; the first-known Christian architect.
Aloisius, of Padua.	5th	Assisted in the erection of the celebrated rotunda at Ravenna, the cupola of which is said to have been of one stone, thirty-eight feet in diameter and fifteen feet thick.
Anthemius, of Trales, of Lydia.	6th	St. Sophia, at Constantinople.
Saxulphus, Abbot of Peterborough, afterwards made Bishop of Lichfield, of England.	7th	Built the Monastery of Medeshampstede, afterwards called Peterborough.
Egbert, Archbishop of York, of England.	8th	Rebuilt York Cathedral.
Romualdus, of France.	9th	The Cathedral of Rheims, the earliest example of Gothic architecture.

AFTER CHRIST.

Name of Architect.	Century.	Principal Works.
Buschetto, of Dulichium.	10th	The Cathedral, or Duomo, of Pisa, the earliest example of the Lombard ecclesiastical style of architecture. It was built in 1016.
Pietro di Ustamber, of Spain.	10th	Cathedral of Chartres.
Lanfranc, Archbishop of Canterbury, of England.	10th	Choir of Canterbury Cathedral, burnt in 1174.
Remigius, Bishop of Lincoln, of England.	11th	Part of Lincoln Cathedral.
Walke'yn, Bishop of Winchester, of England.	11th	Said to have erected the oldest part of Winchester Cathedral.
Mauritius, Bishop of London, of England.	12th	Built old St. Paul's in 1033.
Alexander, Bishop of Lincoln, of England.	12th	Rebuilt Lincoln Cathedral.
Dioti Salvi, of Italy.	12th	Baptistery of Pisa, near the Campo Santo. His works were in the Lombard style and were overloaded with minute ornaments.
Buono, of Venice.	12th	The Tower of St. Mark at Venice, which is three hundred and thirty feet high and forty feet square, built in 1154; a design for enlarging the Church of Santa Maria Maggiore, at Florence, of which the master-walls still exist; the Vicaria and the Castello del' Novo, at Naples; Church of St. Andrew, at Pistola; la Casa della Citta; Campanile at Arezzo.
Wilhelm, or Guglielmo, of Germany.	12th	The Leaning Tower of Pisa built in 1174. Bonnano and Tomaso, two sculptors of Pisa, were also engaged upon it.
William, of Sens, of England.	12th	Canterbury Cathedral.
Peter, of Colechurch, of England.	13th	Began London Bridge.
Robert, of Lusarches, of France.	13th	Cathedral of Amiens, which was continued by Thomas de Cormont and finished by his son Renauld.
Poore, Bishop of Salisbury, of England.	13th	Began Salisbury Cathedral.
Pietro Perez, of Spain.	13th	The Cathedral of Toledo.
Robert de Courcy, of France.	13th	Rebuilt the Cathedral at Rheims.
Juan Rari, of France.	14th	Finished the building of the Church of Notre Dame, of Paris.

AFTER CHRIST.

Name of Architect.	Century	Principal Works.
Rafaelle d'Urbino, of Urbino.	16th	Continued the erection of St. Peter's at Rome after the death of Bramante, his master in architecture; engaged on the buildings of the Farnese Palace; Church of Santa Maria, in Navicella, repaired and altered; stables of Agostino, near the Palazzo Farnese; Palazzo Caffarelli, now Stoppani; the gardens of the Vatican; the façade of the Church of San Lorenzo, and of the Palazzo Uggoccioni, now Pandolfini, at Florence.
Bolton, W., Prior of St. Bartholomew's, of England.	16th	Supposed to have designed Henry VII.'s Chapel, where he was master of the works.
Giovanni Gil de Hontanon, of Spain	16th	Plan of the Cathedral of Salamanca, etc.
Michael Angelo di Buonarotti, of Florence.	16th	Library of the Medici, generally called the Laurentian Library, at Florence; model for the façade of the Church of San Lorenzo, commonly called the Capella dei Depositi; Church San Giovanni, which he did not finish; fortifications at Florence and at Monte San Miniato; monument of Julius II., in the Church of San Pietro in Vincoli, at Rome; plan of the Campidoglio, Palace of the Conservatori, building in the centre, and the flight of steps in the Campidoglio, or Capitol, at Rome; continuation of the Palace Farnese and several gates at Rome, particularly the Porta Nomentana or Pia; steeple of St. Michaele, at Ostia; the gate to the Vineyard de Patriarea Grimani; Tower of S. Lorenzo, at Ardea; Church of Santa Maria, in the Certosa, at Rome; many plans of palaces, churches, and chapels. He was employed on St. Peter's after the death of San Sallo.
Martino de Gainza, of Spain.	16th	The Chapel Royal at Seville.
Machuca, of Spain.	16th	Royal Palace of Granada.
Theodore Havens, of England.	16th	Caius College, Cambridge. A good specimen of the architecture of the day.

AFTER CHRIST.

Name of Architect.	Century.	Principal Works
Carlo Maderno, of Lombardy.	16th	Altered Michael Angelo's design for St. Peter's at Rome from a Greek to a Latin cross; began the palace of Urban VIII.
Sir H. Watton, of England.	17th	Author of "The Elements of Architecture," published in London in 1624.
Inigo Jones, of England.	17th	Banqueting House; chapel, Lincoln's Inn; Surgeon's Hall; arcade, Covent Garden, London; and a vast number of other important works.
Claude Perrault, of France.	17th	Façade of the Louvre, Chapel of Sceaux, Chapel of Notre Dame in the Church of the Petits Pères.
Sir Christopher Wren, of England.	17th	St. Paul's; planned the city of London after the fire, nearly all the churches therein, Hampton Court, etc.
Jules Hardouin Mansard, of France.	17th	The dome of the Hôtel des Invalides. Gallerie du Palais Royal, the Place de Louis de Grand, des Victoires, etc. He was the nephew of François Mansard, the reputed inventor of the Mansard roof.
Alexander Jean Baptiste le Blond, of France.	18th	L'Hôtel de Vendôme, in the Rue d'Enfer, at Paris. He was employed much in Russia by Peter the Great.
Galli da Bibbiena, of Italy.	18th	Theatre at Verona, theatre at Vienna; author of two books on architecture.
James Gibbs, of Scotland.	18th	Radcliffe's Library, Oxford; the new church in the Strand; St.-Martin's-in-the-Fields; King's College, Royal Library, and Senate House, Cambridge.
Sir William Chambers, of England.	18th	Somerset House and many other works; author of a treatise on civil architecture.
Robert Adam, of Scotland.	18th	Architect to George III.; author of a work on the ruins of Spalatro His principal works are the Register Office at Edinburgh, infirmary at Glasgow, the Edinburgh University, Luton House, Adelphi Terrace.
Sir John Soame, of England.	18th	Bank of England, Board of Trade, State-Paper Office.
Charles Percier, of France	18th	Architect of the Tuileries: restorations, etc., at Louvre and Tuileries.

After Christ.

Name of Architect.	Century.	Principal Works.
James Essex, of England.	18th	The earliest, in modern times, who practised solely mediæval art; restoration of Ely and other cathedrals; alterations at various colleges at Cambridge and Oxford.
James Wyatt, of England.	18th	The Pantheon Assembly rooms, palace at Kew, Fonthill Abbey, Doddington Hall, Ashridge House, and many restorations.
Augustus Pugin, of England.	18th	Published "Specimens of Gothic Architecture," "Examples of Gothic Architecture," "Antiquities of Normandy," and other works.
John Nash, of England.	19th	Brighton Pavilion, Haymarket Theatre, Buckingham Palace, Regent's Park and its terraces of dwellings, Regent Street and the Quadrant improvements.
Thomas Rickman, of England.	19th	New court of St. John's College, Cambridge; restoration of the Bishop of Carlisle's palace, Cumberland; upwards of twenty-five churches in the midland counties, several private dwellings. Published "Attempt to Discriminate the Styles of Architecture in England."
Carl Friedrich Schinkel, of Prussia.	19th	Hauptwache Theatre and Museum, Werder-Kirche (Gothic), Bauschule and Observatory at Berlin, theatre at Hamburg, Schloss Krzescowice, Charlottenhof, and the Nicolai-Kirche at Potsdam. Published his designs, many of which were not executed.
Guillaume Abel Blouet, of France.	19th	Published supplement to Roudelet's "L'Art de Bâtir," and revised the tenth edition of that work.
Ernst Friedrich Zwirner, of Prussia.	19th	Restoration of Cologne Cathedral, church at Remagen.
David Hamilton, of Scotland.	19th	The Nelson Monument, the Royal Exchange, the Western Club-house, and other buildings at Glasgow; Hamilton Palace and Lennox Castle, Scotland.
Mr. Joseph Gwilt.	19th	Compiler of the "Encyclopædia of Architecture."

AFTER CHRIST.

Name of Architect.	Century.	Principal Works.
James Fergusson, d. Jan., 1886.	19th	Author of the "History of Architecture."
John Henry Parker, b. in London, 1806; d. Jan. 31, 1884.	19th	Author of the "Glossary of Architecture," "The Domestic Architecture of the Middle Ages," a revised edition of Rickman's "Gothic Architecture."
George Edmund Street.	19th	The Law Courts, London.
William Burges.	19th	Cork Cathedral, restoration of Cardiff Castle.
Sir Gilbert Scott.	19th	Hamburg Cathedral, Edinburgh Cathedral, the Albert Memorial, Midland Station and Hotel at St. Pancras, England.

LIST OF NOTED AMERICAN ARCHITECTS.

CHARLES BULFINCH, the first New England architect, b. 1763, d. 1844. Designed the first theatre in Boston, 1793; the Mass. State House, 1795; the first Catholic church in Boston, 1803; Faneuil Hall, enlarged, 1808; University Hall at Harvard College, 1814; the McLean Asylum at Somerville, 1792–1817, and the Mass. General Hospital, 1818. Architect of the Capitol at Washington from 1797–1818.

JOHN HAVILAND, b. 1792, d. 1825.

Principal works: Pittsburgh Penitentiary; Eastern Penitentiary at Cherry Hill; Hall of Justice, New York; Naval Asylum, Norfolk; New Jersey State Penitentiary; and many other jails, asylums, and public halls.

JONATHAN PRESTON, b. 1801, d. July, 1884; practised in Boston, Mass.

Principal works: The first building of the Massachusetts Institute of Technology, and the building of the Boston Society of Natural History.

WILLIAM WASHBURN, b. in Lyme, N. H., 1808, d. in Boston, November 8, 1890; practised in Boston.

Principal works: The Fifth Avenue and Victoria Hotels in New York, and the Parker House, Tremont House, Revere House, Adams House, Young's Hotel, and the American House in Boston; the Tremont Temple, Boston; Charlestown City Hall, and many other public and private buildings.

Thomas Ustick Walter, LL.D., b. 1804, d. October 30, 1887; practised in Philadelphia, Pa.; was one of the original members of the American Institute of Architects, and president for many years; received the degree of LL.D. from Harvard University, being the first architect to receive that degree in this country.

Principal works: The five original buildings of Girard College designed in 1833 and completed in 1847. Extension of the National Capitol, 1851–65; also the extensions of the Patent Office, Treasury and Post-office buildings, the dome on the old Capitol, the Congressional Library, and the Government Hospital for the Insane; also numerous other buildings of lesser importance. Mr. Walter was a member of the Franklin Institute and of many literary and scientific associations.

Arthur Gilman; practised in New York and Boston, in partnership with Mr. Bryant.

Principal works: Boston City Hall; First Church, on Arlington Street, Boston, and numerous dwelling-houses in New York and Boston. In association with Mr. Edward Kendall, designed the Equitable Life Assurance Company's building on Broadway, New York.

R. G. Hatfield, b. in Elizabeth, N. J., 1815, d. February, 1879; author of the *American House Carpenter* and *Transverse Strains;* associated for thirty-five years with his brother, Oliver P. Hatfield. The firm became widely known as experts and consulting architects in matters pertaining to building construction.

Principal works: House of Refuge, Randall's Island, N. Y.; Westchester County Buildings, White Plains, N. Y.; New York Institution for the Deaf and Dumb, Seaman's Bank for Savings, City Bank building, Security Insurance Co. Building, all of New York City.

Oliver P. Hatfield, d. April, 1891.

John McArthur, Jr., b. in Scotland in 1823, d. January, 1890; practised in Philadelphia, Pa.

Principal works: House of Refuge, Continental Hotel, Girard House, Public Ledger Building, First National Bank Building, the Assembly Building, the Broad Street Presbyterian Church, and the City Hall, all of Philadelphia. Also the Hospital for the Insane, at Warren, Pa.; Lafayette College, Easton, Pa.; and numerous other public and private buildings in Pennsylvania and other States. Was twice tendered the position of Supervising Architect to the United States Government, but declined.

Ebenezer L. Robert, b. 1825; practised in New York City.

Principal works: Standard Oil Company's Building, on Broadway; the Ninth National Bank; the Baptist Church of the Epiphany, on Madison Avenue; St. Paul's Methodist Church, on Fourth Avenue, all of New York City; and the Phœnix Insurance Company's Building, Brooklyn, N. Y.

Alexander R. Esty, b. 1827, d. July 2, 1881; practised in Boston.

Principal works: Union Congregational Church, Boston; Harvard Street Baptist Church, Cambridge, Mass.; Grace Church, Newton, Mass.; Emanuel Church, on Newbury Street, Boston; Buildings of the Colby University, Waterville, Me.; Massachusetts State Normal Schools, at Framingham and Worcester, and the University of Rochester, N. Y.

Carl Pfeiffer, b. in Germany, d. May, 1888; practised in New York City.

Principal works: Fifth Avenue Presbyterian Church, New York; Fifth Avenue Riding School, New York; and many private houses, apartment houses, hotels, etc.

Charles Dexter Gambrill, b. 1832, d. September 13, 1880; practised in New York, first in partnership with Mr. George B. Post, later with H. H. Richardson.

John H. Sturgis; practised in Boston, Mass., with Mr. Charles Brigham as Sturgis & Brigham.

Principal works: Boston Museum of Fine Arts, building of the Boston Young Men's Christian Association, Church of the Advent, residence of Mr. F. L. Ames, and many other fine residences in Boston and vicinity.

A. B. Mullett, b. 1834, d. October 20, 1890; supervising architect to the Trea ury from 1865 to 1875. Also engineer of the District of Columbia for several years. The Post-office buildings in New York, Boston, Cincinnati. St. Louis, and Chicago were designed by him, and also the State, War, and Navy Buildings in Washington.

Henry Hobson Richardson, b. in Louisiana in 1838 or 1839, d. in Brookline, Mass., April, 1886. Graduated at Harvard University in 1859, studied seven years at the Ecole des Beaux-Arts in Paris. Was associated for a short time with Charles D. Gambrill of New York.

A complete list of the works executed by him, arranged in chronological order, may be found in the thirteenth edition of this book.

Perhaps the best-known examples of his work are:

Trinity Church and Brattle Street Church, Boston; City Hall, Albany, and portions of the New York State Capitol; the Library and Town Hall, at North Easton, Mass.; Sever Hall and New Law School, Cambridge, Mass.; County Court House and Jail, Pittsburgh, Pa.; wholesale warehouse for Marshall Field & Co., Chicago; Chamber of Commerce, Cincinnati, Ohio.

THOMAS WISEDELL, b. in England in 1846, d. in New York, July 31, 1884. Educated in the office of Mr. R. J. Withers of London. Associated with Mr. Kimball of New York. Principal works: Madison Square Theatre, and the "Casino," both in New York City

JOSEPH MORRILL WELLS, b 1853, d. in New York, February, 1890. Mr. Wells was a junior partner in the firm of McKim, Mead & White, architects, of New York. The movement of American architects towards the Italian Renaissance, which commenced about the year 1889, was undoubtedly caused more by his influence than that of any other single individual. Among the buildings of the firm, more especially designed by him, are: the Villard houses on Madison Avenue, New York; the "Memorial Building" in New Britain, Conn.; façade of the Century Club, New York, and a fountain in Portland, Oregon.

HENRY O. AVERY, d. 1890; studied at the School of Fine Arts in Paris. Took an important part in designing the houses of W. K. Vanderbilt and Henry G. Marquand; a prominent member of the architectural League of New York, the Archæological Institute, and the Society of American Artists.

JOHN WELLBORN ROOT, b. in Georgia, January 10, 1850, d. in Chicago, Ill., January 15, 1891. Entered into partnership with Daniel H. Burnham in 1873, which continued until his death. Mr. Root was the designer of the firm. They designed and executed seventy-seven public buildings, many of them of the first class, and one hundred and twenty residences. Of their public buildings the following were perhaps the most important: Calumet Club House, Art Institute, Academy of Fine Arts, Montauk Block, Calumet Building, Rialto Office Building, Insurance Exchange Building, Grannis Block, Phœnix Building, The Rookery, Masonic Building, Woman's Temple, First Regiment Armory, all of Chicago; the Mills Block, San Francisco; Midland Hotel, Board of Trade Building, American National Bank Building of Kansas City. Mr. Root was

secretary of the American Institute of Architects at the time of his death.

Herbert C. Burdett, b. in Boston, 1855, d. in Buffalo, April 10, 1891; associated with J. Herbert Marling, as Marling & Burdette, and practised in Buffalo, N. Y. Principal works: The Saturn Club House and numerous fine residences in Buffalo.

George Washingtor Percy, A.A.I.A., b. at Bath, Me. July 5, 1847, d. during 1900. Practiced in San Francisco, California, 1876–1900; from 1879 associated with Mr. F. F. Hamilton. The firm designed many important buildings in and about San Francisco and Los Angeles, also at Honolulu, H. I. President of the Technical Society of the Pacific Coast, 1898–1900.

Dankmark Adler, F.A.I.A., b. in Langsfeld, Saxe-Weimar, July 3, 1844. Practised architecture in Chicago from 1869 until his death, April 16, 1900. Was for many years associated with Louis H. Sullivan, Mr. Adler being the "practical man" of the firm; secretary A.I.A. 1891–92; member Board of Directors 1890–93.

Edward C. Cabot, F.A.I.A., b. in Boston April, 1818, d. January, 1901. Practised in Boston. For a number of years associated with Mr. F. W. Chandler. Designed the Boston Athenæum in 1846, the Boston Theatre in 1852–53, and in association with Mr. Chandler, the Johns Hopkins Hospital, Baltimore. Became a member of the A.I.A. in 1857, and was president of the Boston Chapter for thirty-three years.

Edward Hale Kendall, F.A.I A., b. in Boston, July 30, 1842, d. March 10, 1901. Practised in New York from about 1868 until his death. His chief works are perhaps the first plans of the Equitable Building, the Field Building, No. 1 Broadway, the Methodist Book Concern, the Goelet houses, and the Washington Bridge, of which he was the consulting architect. Was vice-president A.I.A. in 1885, a director for many years, and president in 1892 and 1893. Was president of the New York Chapter from 1884–88.

Napoleon Eugene, H. C. Le Brun, F.A.I.A., b. in Philadelphia January 2, 1821, d. July 9, 1901. Practised in Philadelphia 1842–65, when he removed to New York. Among the prominent buildings which he designed in Philadelphia are the Cathedral, the Academy of Music, the old Tabernacle Presbyterian Church, the Girard Estate Building, and several county buildings and prisons. In New York City, in connection with

his son, he erected many dwellings and public buildings, including the Masonic Temple, several large and beautiful churches, the New York Foundling Asylum, the Metropolitan Insurance Building on Madison Square, the Home Life Insurance Building, and several municipal edifices Member A.I.A. from 1868 until his death, twice president of the New York Chapter, and also president of the Willard Architectural Commission.

EDWIN CLARK, F.A.I.A., b. in Philadelphia August 15, 1822, d. January 6, 1902. Architect of the United States Capitol from 1865 until his death.

JAMES BROWN LORD, F.A.I.A., b. in New York 1859, d. June 1, 1902. Designed the Delmonico Building, New York; the Bloomingdale Asylum at White Plains; the Carnegie Library in East Seventy-sixth Street; and the Appellate Court Building on Madison Avenue and Twenty-fourth Street.

WALTER COPE, F.A.I.A., b. in Philadelphia October 30, 1860, d. November 3, 1902. Associated with John Stewardson and E. L. Stewardson from 1885 until his death.

Among the notable buildings designed by this firm are Denbigh, Pembroke, and Rockefeller Halls, all dormitories of Bryn Mawr College; the Dormitories, Law School, and Medical Laboratories of the University of Pennsylvania; Blair Hall, Stafford-Little Hall, and Gymnasium of Princeton University; the Pennsylvania Institution for the Instruction of the Blind, at Overbrook, Pa.; the Washington University of St. Louis, Mo.; the City Hall, at Atlantic City, N. J.; the Harrison Office Building and the Harrison stores in Philadelphia, and many fine residences.

HENRY VAN BRUNT, F.A.I.A., b. in Boston, Sept. 5, 1832, d. April 6, 1903. Student of Richard M. Hunt, practised in Boston, under the firm name of Ware & Van Brunt, until 1882, when Mr. Ware accepted the chair of Architecture at Columbia College, and Mr. Van Brunt formed a new partnership with Mr. Frank M. Howe. The firm of Van Brunt & Howe moved to Kansas City in 1887, and existed until the death of Van Brunt. President of the American Institute of Architects, 1899, and a writer of great ability.

Notable buildings designed by Ware and Van Brunt: Memorial Hall of Harvard College; First Church of Boston; St. Stephens Church, Lynn, Mass.; buildings for Wellesley College. Of Van Brunt & Howe: New Coates House; the Gibraltar

Building; Emery-Bird-Thayer Building Kansas City Star Building, all of Kansas City; the Union Depot, Denver.

Bruce Price, F.A.I.A., b. in Cumberland, Md., 1845, d. in Paris, May, 1903. An architect of great brilliance and originality. His chief works are the American Surety Company's Building, N. Y.; St. James Building, N. Y.; the group of buildings near Lakewood, N. J., which he designed for Mr. George J. Gould; Osborn Hall at Yale University, and a picturesque hotel in Quebec known as the Château Frontenac. Was for some years president of the N. Y. Architectural League.

Professional Practice of Architects, and Schedule of Usual and Proper Minimum Charges.

(A. I. A. Schedule, revised October, 1903.)

The architect's professional services consist in making the necessary preliminary studies, working drawings, specifications, large-scale and full-size details, and in the general direction and supervision of the work, for which the minimum charge is five per cent. upon the cost of the work.

For new buildings costing less than $10,000, and for furniture, monuments, decorative and cabinet work, it is usual and proper to charge a special fee in excess of the above.

For alterations and additions to existing buildings the fee is ten per cent. upon the cost of the work.

Consultation fees for professional advice are to be paid in proportion to the importance of the questions involved.

None of the charges above enumerated covers alterations and additions in contracts, drawings, and specifications, nor professional or legal services incidental to negotiations for site, disputed party walls, right of light, measurement of work, or failure of contractors. When such services become necessary, they shall be charged for according to the time and trouble involved.

Where heating, ventilating, mechanical, electrical, and sanitary problems in a building are of such a nature as to require the assistance of a specialist, the owner is to pay for such assistance. Chemical and mechanical tests, when required, are to be paid for by the owner.

Necessary travelling expenses are to be paid by the owner.

Drawings and specifications, as instruments of service, are the property of the architect.

The architect's payments are due as his work progresses in the following order: Upon completion of the preliminary sketches one-fifth of the entire fee; upon completion of working drawings and specifications, two-fifths; the remaining two-fifths being due from time to time in proportion to the amount of work done by the architect in his office and at the building.

Until an actual estimate is received, the charges are based upon the proposed cost of the work, and payments are received as instalments of the entire fee, which is based upon the actual cost to the owner of the building or other work, when completed, including all fixtures necessary to render it fit for occupation.

The architect is entitled to extra compensation for furniture or other articles purchased under his direction.

If any material or work used in the construction of the building be already upon the ground or come into the owner's possession without expense to him, its value is to be added to the sum actually expended upon the building before the architect's commission is computed.

In case of the abandonment or suspension of the work, the basis of settlement is as follows: Preliminary studies, a fee in accordance with the character and magnitude of the work; preliminary studies, working drawings, and specifications, three-fifths of the fee for complete services.

The supervision of an architect (as distinguished from the continuous personal superintendence which may be secured by the employment of a clerk of the works) means such inspection by the architect or his deputy of work in studios and shops, or of a building or other work in process of erection, completion, or alteration, as he finds necessary to ascertain whether it is being executed in conformity with the drawings and specifications or directions. He is to act in constructive emergencies, to order necessary changes, and to define the true intent and meaning of the drawings and specifications, and he has authority to stop the progress of the work and order its removal when not in accordance with them.

On buildings where the constant services of a superintendent are required, a clerk of the works shall be employed by the architect at the owner's expense.

CONTRACT BETWEEN ARCHITECT AND OWNER.

From.................................., Architect,
to.., Owner.
For a compensation of......................................
the architect proposes to furnish preliminary sketches, contract working drawings and specifications, detail drawings and general superintendence of building operations, and, also, to audit all accounts, for a...
to be erected for...
on...

Terms of payment to be as follows:

One fifth when the preliminary sketches are completed; three tenths when the drawings and specifications are ready for letting

contracts; thereafter at the rate of......per cent. upon each certificate due to the contractor..............................

If work upon the building is postponed or abandoned, the compensation for the work done by the architect is to bear such relation to the compensation for the entire work as determined by the published schedule of fees of the American Institute of Architects.

In all transactions between the owner and contractor, the architect is to act as the owner's agent, and his duties and liabilities in this connection are to be those of agent only.

A representative of the architect will make visits to the building for the purpose of general superintendence, of such frequency and duration as, in the architect's judgment, will suffice, or may be necessary to fully instruct contractors, pass upon the merits of material and workmanship, and maintain an effective working organization of the several contractors engaged upon the structure.

The architect will demand of the contractors proper correction and remedy of all defects discovered in their work, and will assist the owner in enforcing the terms of the contracts; but the architect's superintendence shall not include liability or responsibility for any breach of contract by the contractors.

The amount of the architect's compensation is to be reckoned upon the total cost of the building, including all stationary fixtures.

Drawings and specifications are instruments of service, and as such are to remain the property of the architect.

.........................., Architect.

Approved and accepted..............., 190

..............................., Owner.

THE UNIFORM CONTRACT.*

Form of Contract Adopted and Recommended for General Use by the American Institute of Architects and the National Association of Builders. Revised 1902.

THIS AGREEMENT, made the..in the year one thousand nine hundred and............by and between...party of the first part (hereinafter designated the Contractor), and...party of the second part (hereinafter designated the Owner),

WITNESSETH that the Contractor , in consideration of the agreements herein made by the Owner , agree with the said Owner as follows:

ARTICLE I. The Contractor shall and will provide all the materials and perform all the work for the.. as shown on the drawings and described in the specifications prepared by...Architect , which drawings and specifications are identified by the signatures of the parties hereto, and become hereby a part of this contract.

ART. II. It is understood and agreed by and between the parties hereto that the work included in this contract is to be done under the direction of the said Architect , and that...... decision as to the true construction and meaning of the drawings and specifications shall be final. It is also understood and agreed by and between the parties hereto that such additional drawings and explanations as may be necessary to detail and illustrate the work to be done are to be furnished by said Architect , and they agree to conform to and abide by the same so far as they may be consistent with the purpose and intent of the original drawings and specifications referred to in Art. I.

It is further understood and agreed by the parties hereto that any and all drawings and specifications prepared for the purposes of this contract by the said Architect are and remain....... property, and that all charges for the use of the same, and for the services of said Architect are to be paid by the said Owner .

ART III. No alterations shall be made in the work except upon written order of the Architect ; the amount to be paid by the Owner or allowed by the Contractor by virtue of such alterations to be stated in said order. Should the Owner and Contractor not agree as to amount to be paid or allowed, the work shall go on under the order required above, and in case of failure

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to agree, the determination of said amount shall be referred to arbitration, as provided for in Art. XII of this contract.

Art. IV. The Contractor shall provide sufficient, safe and proper facilities at all times for the inspection of the work by the Architect or........ authorized representatives; shall, within twenty-four hours after receiving written notice from the Architect to that effect, proceed to remove from the grounds or buildings all materials condemned by.......... whether worked or unworked, and to take down all portions of the work which the Architect shall by like written notice condemn as unsound or improper, or as in any way failing to conform to the drawings and specifications, and shall make good all work damaged or destroyed thereby.

Art. V. Should the Contractor at any time refuse or neglect to supply a sufficiency of properly skilled workmen, or of materials of the proper quality, or fail in any respect to prosecute the work with promptness and diligence, or fail in the perform ance of any of the agreements herein contained, such refusal, neglect or failure being certified by the Architect , the Owner shall be at liberty, after.............. days' written notice to the Contractor , to provide any such labor or materials, and to deduct the cost thereof from any money then due or thereafter to become due to the Contractor under this contract; and if the Architect shall certify that such refusal, neglect or failure is sufficient ground for such action, the Owner shall also be at liberty to terminate the employment of the Contractor for the said work and to enter upon the premises and take possession, for the purpose of completing the work included under this contract, of all materials, tools and appliances thereon, and to employ any other person or persons to finish the work, and to provide the materials therefor; and in case of such discontinuance of the employment of the Contractor .... shall not be entitled to receive any further payment under this contract until the said work shall be wholly finished, at which time, if the unpaid balance of the amount to be paid under this contract shall exceed the expense incurred by the Owner in finishing the work, such excess shall be paid by the Owner to the Contractor ; but if such expense shall exceed such unpaid balance, the Contractor shall pay the difference to the Owner . The expense incurred by the Owner as herein provided, either for furnishing materials or for finishing the work, and any damage incurred through such default, shall be audited and certified by the Architect , whose certificate thereof shall be conclusive upon the parties.

Art. VI. The Contractor shall complete the several portions, and the whole of the work comprehended in this agreement by and at the time or times hereinafter stated, to wit:

....

....

....

Art. VII. Should the Contractor be delayed in the prosecution or completion of the work by the act, neglect or default of the Owner , of the Architect , or of any other contractor

employed by the Owner upon the work, or by any damage caused by fire, lightning, earthquake, cyclone or other casualty for which the Contractor ....not responsible, or by strikes or lockouts caused by acts of employes, then the time herein fixed for the completion of the work shall be extended for a period equivalent to the time lost by reason of any or all the causes aforesaid, which extended period shall be determined and fixed by the Architect ; but no such allowance shall be made unless a claim therefor is presented in writing to the Architect within forty-eight hours of the occurrence of such delay.

Art. VIII. The Owner agree to provide all labor and materials essential to the conduct of this work not included in this contract in such manner as not to delay its progress, and in the event of failure so to do, thereby causing loss to the Contractor , agree that.........will reimburse the Contractor for such loss; and the Contractor agree that if..... shall delay the progress of the work so as to cause loss for which the Owner shall become liable, then.........shall reimburse the Owner for such loss. Should the Owner and Contractor fail to agree as to the amount of loss comprehended in this Article, the determination of the amount shall be referred to arbitration as provided in Art. XII of this contract.

Art. IX. It is hereby mutually agreed between the parties hereto that the sum to be paid by the Owner to the Contractor for said work and materials shall be.................

....

subject to additions and deductions as hereinbefore provided, and that such sum shall be paid by the Owner to the Contractor , in current funds, and only upon certificates of the Architect , as follows:

....

....

The final payment shall be made within..................... days after the completion of the work included in this contract, and all payments shall be due when certificates for the same are issued.

If at any time there shall be evidence of any lien or claim for which, if established, the Owner of the said premises might become liable, and which is chargeable to the Contractor , the Owner shall have the right to retain out of any payment then due or thereafter to become due an amount sufficient to completely indemnify.........against such lien or claim. Should there prove to be any such claim after all payments are made, the Contractor shall refund to the Owner all moneys that the latter may be compelled to pay in discharging any lien on said premises made obligatory in consequence of the Contractor default.

Art X. It is further mutually agreed between the parties hereto that no certificate given or payment made under this contract, except the final certificate or final payment, shall be conclusive evidence of the performance of this contract, either

wholly or in part, and that no payment shall be construed to be an acceptance of defective work or improper materials.

ART. XI. The Owner shall during the progress of the work maintain insurance on said work, in......own name and in the name of the Contractor , against loss or damage by fire, lightning, earthquake, cyclone or other casualty. The policies to cover all work incorporated in the building, and all materials for the same in or about the premises, and shall be made payable to the parties hereto, as their interest may appear.

ART. XII. In case the Owner and Contractor fail to agree in relation to matters of payment, allowance or loss referred to in Arts. III or VIII of this contract, or should either of them dissent from the decision of the Architect referred to in Art. VII of this contract, which dissent shall have been filed in writing with the Architect within ten days of the announcement of such decision, then the matter shall be referred to a Board of Arbitration consisting of..........................
..................................in behalf of the Owner ,
and...........................in behalf of the Contractor ,
these two to select a third. The decision of any two of this Board shall be final and binding on both parties hereto. In event of the death or inability to serve of the party named in behalf of the Owner , then the Owner shall select a person in his place; in event of the death or inability to serve of the party named in behalf of the Contractor , then the Contractor shall select a person in his place; in event of the death or inability to serve of the third party, then the remaining arbitrators shall choose a person in his place. Each party hereto shall pay one-half of the expense of such reference.

ART. XIII. The said parties for themselves, their heirs, successors, executors, administrators and assigns, do hereby agree to the full performance of the covenants herein contained.

IN WITNESS WHEREOF, the parties to these presents have hereunto set their hands and seals, the day and year first above written.

In Presence of

ARCHITECTS' LICENSE LAW—STATE OF ILLINOIS.

TO PROVIDE FOR THE LICENSING OF ARCHITECTS AND REGULATING THE PRACTICE OF ARCHITECTURE AS A PROFESSION.

AN ACT

Enacted by the Fortieth General Assembly at the Regular Biennial Session, Approved June 3, 1897, *and in Force July* 1, 1897; *with Amendments Adopted by the Forty-first General Assembly and Approved April* 19, 1899. *In Force July* 1, 1899.

APPOINTMENT OF A STATE BOARD OF EXAMINERS OF ARCHITECTS

SECTION 1. *Be it enacted by the people of the State of Illinois, represented in General Assembly:* That within thirty days after the passage ot this act the Governor of this State shall, by the

advice and consent of the Senate, appoint a State Board of Examiners of Architects, to be composed of five members, one of whom shall be a member of the faculty of the Illinois State University, and the other four shall be architects residing in the State of Illinois, who have been engaged in the practice of architecture at least ten years. Two of the said practicing architects appointed as examiners shall be designated to hold office for two years from the date of the passage of this act, and the other two, together with the member of the faculty aforesaid, shall hold office for four years from the passage of this act; and thereafter upon the expiration of the term of office of the person so appointed, the Governor of the State shall appoint a successor to each person whose term of office shall expire, to hold office for four years, and said person so appointed shall have the above specified qualifications. In case appointment of a successor is not made before the expiration of the term of any member, such member shall hold office until his successor is appointed and duly qualified. Any vacancy occurring in membership of the board shall be filled by the Governor of the State for the unexpired term of such membership.

[Sections 2 and 3 relate to the organization of the board, salaries, meetings, etc.]

EXAMINATIONS—FEES.

SEC. 4. Provisions shall be made by the board hereby constituted for holding examinations at least twice in each year, of applicants for license to practice architecture, and any person over twenty-one years of age, upon payment of a fee of **fifteen dollars** to the secretary of the board, shall be entitled to an examination for determining his or her qualifications. All examinations shall be made directly by said board, or a committee of two members delegated by the board, and due notice of the time and place of the holding of such examinations shall be published, as in the case provided for the publication of the rules and regulations thereof. The examination shall have special reference to the construction of buildings, and a test of the knowledge of the candidate of the strength of materials, and of his or her ability to make practical application of such knowledge in the ordinary professional work of an architect, and in the duties of a supervisor of mechanical work on buildings, and should also seek to determine his or her knowledge of the laws of sanitation as applied to buildings. If the result of the examination of any applicant shall be satisfactory to a majority of the board, under its rules, the secretary shall upon an order of the board, issue to the applicant a certificate to that effect, and upon payment to the secretary of the board by the candidate of a fee of **twenty-five dollars,** he shall thereupon issue to the person therein named a license to practice architecture in the State, in accordance with the provisions of this act, which license shall contain the full name, birth-place and age of the applicant, and be signed by the president and secre-

tary, and sealed with the seal of the board. If an applicant fails to pass said examination, his or her fee shall be returned.

All papers received by the secretary in relation to applications for license shall be kept on file in his office, and a proper index and record thereof shall be kept by him.

ARCHITECTS WHO ARE ENTITLED TO LICENSE WITHOUT AN EXAMINATION.

SEC. 5. Any person who shall, by affidavit, show to the satisfaction of the State Board of Examiners of Architects that he or she was engaged in the practice of the profession of architecture on the date of the passage of this act shall be entitled to a license without an examination, provided such application shall be made within six months after the passage of this act. Such license, when granted, shall set forth the fact that the person to whom the same was issued was practicing architecture in this State at the time of the passage of this act, and is therefore entitled to a license to practice architecture without an examination by the board of examiners, and the secretary of the board shall, upon the payment to him of the fee of **twenty-five dollars,** issue to the person named in said affidavit, a license to practice architecture in this State, in accordance with the provisions of this act. In the case of a co-partnership of architects, each member whose name appears must be licensed to practice architecture. No stock company or corporation shall be licensed to practice architecture, but the same may employ licensed architects. Each licensed architect shall have his or her license recorded in the office of the county clerk in each and every county in this State in which the holder thereof shall practice, and he or she shall pay to the clerk the same fee that is charged for the recording of notarial commissions. A failure to have his or her license so recorded shall be deemed sufficient cause for revocation of such license.

COUNTY CLERKS TO KEEP RECORD OF LICENSES RECORDED.

SEC. 6. Each county clerk shall keep in a book, provided for the purpose, a complete list of all licenses recorded by him under the provisions of this act, together with the date of the issuance of each license.

LICENSED ARCHITECTS TO HAVE A SEAL.

SEC. 7. Every licensed architect shall have a seal, the impression of which must contain the name of the architect, his or her place of business, and the words, "Licensed Architect," "State of Illinois," with which he shall stamp all drawings and specifications issued from his office, for use in this State.

PENALTY FOR PRACTICING ARCHITECTURE WITHOUT A LICENSE.

SEC. 8. After six months from the passage of this act it shall be unlawful and it shall be a misdemeanor punishable by a fine of not less than \$50 nor more than \$500 for each and every week during which said offense shall continue, for any

person to practice architecture without a license in this State, or to advertise, or put out any sign or card, or other device which might indicate to the public that he or she is entitled to practice as an architect.

PERSONS WHO ARE TO BE REGARDED AS ARCHITECTS.

SEC. 9. Any person who shall be engaged in the planning or supervision of the erection, enlargement, or alteration of buildings for others, and to be constructed by other persons than himself, shall be regarded as an architect within the provisions of this act, and shall be held to comply with the same; but nothing contained in this act shall prevent the draughtsmen, students, clerks of works or superintendents, and other employes of those lawfully practicing as architects, under license as herein provided for, from acting under the instruction, control or supervision of their employers; or shall prevent the employment of superintendents of buildings paid by the owners from acting, if under the control and direction of a licensed architect who has prepared the drawing and specifications for the building. The term building in this act shall be understood to be a structure, consisting of foundations, walls, and roof, with or without the other parts; but nothing contained in this act shall be construed to prevent any person, mechanic or builder from making plans and specifications for, or supervising the erection, enlargement, or alteration of any building that is to be constructed by himself or employes; nor shall a civil engineer be considered as an architect unless he plans, designs and supervises the erection of buildings, in which case he shall be subject to all the provisions of this act, and be considered as an architect.

LICENSE REVOKED.

SEC. 10. Architects' licenses issued in accordance with the provisions of this act shall remain in full force until revoked for cause, as hereinafter provided. Any license so granted may be revoked by unanimous vote of the State Board of Examiners of Architects for gross incompetency, or recklessness in the construction of buildings, or for dishonest practices on the part of the holder thereof; but before any license shall be revoked such holder shall be entitled to at least twenty days' notice of the charge against him, and of the time and place of the meeting of the board for the hearing and determining of such charge. And on the cancellation of such license it shall be the duty of the secretary of the board to give notice of such cancellation to the county clerk of each county in the State in which the license has been recorded, whereupon the clerks of the counties shall mark the license recorded in his office cancelled. After the expiration of six months from the revocation of a license, the person whose license was revoked may have a new license issued to him by the secretary upon certificate of the Board of Examiners, issued by them upon satisfactory evidence of proper reasons for his reinstatement, and upon payment to the secretary of the fee of five dollars.

For the purpose of carrying out the provisions of this act relating to the revocation of licenses, the board shall have the power of a court of record, sitting in the county in which their meeting shall be held, and the power to issue subpœnas and compel the attendance and testimony of witnesses. Witnesses shall be entitled to the same fees as witnesses in a court of record, to be paid in like manner. The accused shall be entitled to the subpœna of the board for his witnesses and to be heard in person or by counsel in open public trial.

RENEWAL OF LICENSE.

SEC. 11. Every licensed architect in this State who desires to continue the practice of his profession shall annually, during the time he shall continue in such practice, pay to the secretary of the board during the month of July a fee of **five dollars** and the secretary shall thereupon issue to such licensed architect a certificate of renewal of his license for the term of one year. Any licensed architect who shall fail to have his license renewed during the month of July in each and every year shall have his license revoked; and it shall be the duty of the secretary of the board to give notice of such revocation to the county clerk in each county in the State, whereupon the clerks of the counties shall make an entry of such revocation accordingly.

But the failure to renew said license in apt time shall not deprive such architect of the right to renewal thereafter; and the secretary of the board shall give like notice of such renewal; but the fee to be paid upon the renewal of license after the month of July shall be ten dollars, to cover the additional expense incurred by the board on account of such notices.

REPORT OF PROCEEDINGS TO BE FILED WITH THE AUDITOR OF PUBLIC ACCOUNTS.

SEC. 12. Within the first week of December, after the organization of the board, and annually thereafter, the secretary of the board shall file with the Auditor of State a full report of the proceedings of the board, and a complete statement of the receipts and expenditures of the board, attested by the affidavits of the president and secretary, subject to the approval of the State Auditor.

COLLEGES AND SCHOOLS OF ARCHITECTURE IN THE UNITED STATES.

Columbia University, New York.—*School of Architecture.* Alfred D. F. Hamlin, Professor in charge. Offers: (1) Full four-year course leading to degree of Bachelor of Science. In the fourth year the student may elect a specialized course in Advanced Architectural Engineering, in place of the usual course in Advanced Design. (2) Advanced courses leading to the degrees of Master of Arts, and Doctor of Philosophy. (3)

Special or elective courses for students not candidates for a degree. Tuition, $200 per year.

Cornell University, Ithaca, N. Y.—*College of Architecture.* Prof. John V. Van Pelt in charge. Prof. Clarence A. Martin, Secretary. Offers: (1) Three courses leading to the degree of *Bachelor of Architecture* as follows: *First,* the regular four-year course; *Second,* a four-year course allowing specialization in Architectural Design; *Third,* a four-year course allowing specialization in Architectural Engineering. (2) A two-year special course in Architecture, leading to a certificate. (3) A regular two-year course in Painting, leading to a certificate. (4) Special courses in Painting, arranged for individual cases but not leading to a certificate or degree. Tuition, $125 per year.

Harvard University, Lawrence Scientific School.—*Department of Architecture.* Herbert Langford Warren, A.M., Nelson Robinson, Jr., Professor of Architecture, in charge. Offers: (1) Full four-year programme of courses in Architecture leading to the degree of *Bachelor of Science* in Architecture. (2) Competent special students are admitted to take a partial course. A certificate will be given to such students. Tuition, $150 per year.

Lawrence Scientific School.—*Department of Landscape Architecture.* Prof. Frederick Law Olmstead, A.B., in charge. Offers: (1) Full four-year programme of courses leading to the degree of *Bachelor of Science* in Landscape Architecture. (2) Competent special students are admitted to take a partial course, to whom a certificate will be given. Tuition, $150 per year.

Massachusetts Institute of Technology, Boston, Mass.—Francis W. Chandler, Professor in charge. Offers: (1) Three courses leading to the degree of *Bachelor of Science: First,* the regular four-year course in Architecture; *Second,* a four-year course allowing specialization in Architectural Engineering; *Third,* a four-year course allowing specialization in Landscape Architecture. (2) Special students are received on the basis of office experience or college graduation and preparation in Geometry and Drawing. Graduate courses lead to the Master's degree. Tuition, $250 per year.

University of Pennsylvania, Philadelphia, Pa.—*Course in Architecture.* Warren Powers Laird, Professor in charge. Offers: (1) Full four-year course leading to the degree of B.S in Architecture. (2) Two-year special course

leading to a certificate of proficiency. (3) Five-year or graduate course leading to the degree of M.S. in Architecture. (4) Combined six-year course in Arts and Architecture leading to the degree of A.B. at the end of the fourth year and B.S. in Architecture at the end of the sixth year. (5) Course in Architectural Engineering leading to the degree of B.S. in Architecture and differentiated from the regular four-year course in Architecture by the substitution in the last year of specialized work in engineering subjects for Architectural Designing, Drawing, etc. Tuition for all courses, $150 per year.

University of Illinois, Urbana, Ill. — *Courses in Architecture.* Nathan Clifford Ricker, Professor in charge. Offers: (1) Full four-year course, leading to degree of B.S. in Architecture. (2) Full four-year course leading to degree of B.S. in Architectural Engineering. Tuition is free to residents of the State. There is an incidental fee of $24 a year.

Ohio State University, Columbus, Ohio.—*Course in Architecture.* J. N. Bradford, Professor in charge. Offers: Full four-year course leading to degree. Tuition, free.

University of California, Oakland, Cal. —Has recently established a Department of Architecture with John Galen Howard, Professor in charge.

Syracuse University, Syracuse, N. Y. — *College of Fine Arts.* F. W. Revels, Professor of Architecture. Offers: (1) Full four-year course leading to degree. (2) Two-year special course leading to certificate of proficiency. Tuition, $120 per year.

Washington University, St. Louis, Mo.—*Course in Architecture.* Frederick M. Mann, Professor in charge. Offers: (1) Four-year course leading to a degree. (2) Special course for draughtsmen. Tuition, $150 per year; special course, $100 per year.

Rose Polytechnic Institute, Terre Haute, Ind.—*Department of Architecture.* Malverd A. Howe, C.E., director. Offers a full four-year course, designed to give a thorough training in Architectural Engineering together with systematic instruction in Architectural Design. Tuition, $100 per year.

Drexel Institute, Philadelphia, Pa.—*School of Architecture.* Arthur Truscott, director. Offers a two-year course in Architecture, a large share of the time being devoted to purely Architectural work. Tuition, $60 a year.

Pratt Institute, Brooklyn, N. Y.—*Course in Architecture.* Walter S. Perry, Director of Department of Fine Arts. Offers: Full two-years' course leading to a certificate of proficiency. Tuition, $45 per year.

Academy of Architecture and Industrial Science, 1742 Chouteau Ave., St. Louis, Mo.—H. Maack Principal. This is a private school founded by Mr. Maack, in 1885, and designed more particularly to meet the wants of building tradesmen, offering them such instruction as is necessary to attain the highest proficiency in their trade, and to fully understand the plans and details of complicated buildings. There is also a special course for those desiring to fit themselves for positions as draughtsmen in architects' offices. Tuition for the regular course is $50 for a three-months' term, or $300 for the full course of eight terms, or $100 for the year. There are several special courses which may be commenced at any time, and for which the tuition varies.

The Society of Beaux-Arts Architects of New York has established a course of study for architectural draughtsmen, modelled on the system adopted by the École des Beaux-Arts, Paris, France. The course is divided into two classes: Class B, into which any one of either sex may enter without any preliminary examination; Class A, which the student reaches after having received certain awards in Class B. On completing the course, which is not limited by time, the Society awards a certificate of proficiency.

Address

CHAIRMAN, Committee on Education,
3 East 33d St., New York.

Instruction in Architecture, Architectural Engineering, and Drawing is also given by the International Correspondence Schools, Scranton, Pa., and by the American School of Correspondence, at Armour Institute of Technology, Chicago, Ill.

TRAVELLING FELLOWSHIPS AND SCHOLARSHIPS.

Rotch Travelling Scholarship.—C. H. Blackall, Secretary, 1 Somerset St., Boston, Mass. Candidates must be under thirty years of age, must have worked during two years in Massachusetts in the employ of an architect resident in Massachusetts, and will be required to pass preliminary examinations upon the following subjects:

I. Construction, Theory and Practice. (Written examination.)

II. An Elementary Knowledge of the French Language. (Written examination.)

III. History of Architecture. (Written examination.)

IV. Freehand Drawing from the Cast.

Candidates who pass in these preliminary examinations will be asked to present themselves later for the competition in Design. The successful candidate in each yearly examination receives from the Trustees of the Scholarship annually, for two years, $1,000 to be expended in foreign travel and study, provided always that the beneficiary shows such fitness and diligence as may be required of him.

The Boston Society of Architects has offered the sum of $75 as a second prize.

The Society of Beaux-Arts Architects, Travelling Scholarship.—Lloyd Warren, Chairman Com. on Education, 3 East 33d St., New York. A fund of $2,000 has been provided to defray the expenses of this prize, which will be awarded July, 1904, and the recipient will spend two years in travel and study abroad. The award will be based on the result of three competitive trials, to which all American draughtsmen under 28 years of age are eligible. The four draughtsmen holding the best averages next to the final winner of the scholarship will be awarded the sum of $100 each.

Columbia University Travelling Fellowships.—Four travelling fellowships have been established, open to all graduates of the School of Architecture under 30 years of age; they are awarded in May of each year.

LIST OF VALUABLE BOOKS FOR ARCHITECTS, DRAUGHTSMEN, AND BUILDERS.

[The author has carefully examined nearly all of the books named below, and can recommend them as containing useful information on the subjects under which they are listed. Name and address of publisher given at end of the list.]

Architecture.

	Price
*Handbook of Architectural Styles.** By A. Rosengarten....	$2.50
History of Architecture.[1] By Prof. A. D. F. Hamlin........	2.00
Vignola. The Five Orders of Architecture.[2] Edited by Arthur Lyman Tuckerman..........................	5.00

Price

Vignola. American edition prepared for Bates & Guild Co. $5.00
The American Vignola.[3] By Prof. William R. Ware. 3.00
*Stepping-stone to Architecture.** By Thomas Mitchell. 0.50
A Discussion of Composition, especially as applied to Architecture. By John V. Van Pelt 2.00
*Handbook of Ornament.** By Meyer 3.60
A Dictionary of Architecture and Building.[5] By Russell Sturgis. In three volumes, per volume. 18.00

Building Construction, Superintendence, and Specifications.

(See also Foundations and Iron and Steel Construction.)

A Handbook for Superintendents of Construction, Architects, Builders, and Building Inspectors.[6] By H. G. Richey. 16mo, 747 pages 4.00
Cements, Limes, and Plasters: Their Materials, Manufacture, and Properties.[6] By Edwin C. Eckel, C.E. 8vo, 754 pp. 6.00
Building Construction and Superintendence.[2] Part I. Masonry and Plastering. Part II. Carpenters' Work. Part III. Trussed Roofs and Roof-Trusses. By F. E. Kidder. Each volume sold separately, per volume. . 4.00
Building Superintendence.[5] By T. M. Clark. 3.00
Safe Building.[5] By Louis De Coppert Berg. Two volumes, each. 5.00
Details of Building Construction.[4] By Prof. Clarence A. Martin. 2.00
Inspectors' Pocket-Book,[6] for the use of Inspectors and Superintendents. By Austin T. Byrne. 3.00
A Practical System for Writing Specifications for Buildings.[2] By W. Frank Bower 5.00

Concrete, Plain and Reinforced.

Experimental Researches on Reinforced Concrete.[13] By Armand Considère. Translated by Leon Moisseiff, C.E. Authorized American Edition. $2.00.

A Treatise on Concrete, Plain and Reinforced.[6] By Frederick W. Taylor, M.E., and Sanford E. Thompson, Assoc. M. Am. Soc. C.E., with chapters by R. Feret, Wm. B. Fuller, and Spencer B. Newberry. 8vo, 620 pp. $5.00.

Reinforced Concrete.[14] By Chas. F. Marsh, Assoc. M. Inst. C.E., Assoc. M. Inst. M. E. 4to, $7\frac{3}{4} \times 11$, 530 pp., 511 ill. $7.00.

Reinforced Concrete.[11] By A. W. Buel and C. S. Hill. $5.00.

Drawing.

Price.

Architectural Drawing. By C. Franklin Edminster. $2.00
[The most practical and complete course in the elements of Architectural Drawing now published.]

Architectural Perspective for Beginners.[2] By F. A. Wright. 3.00

Pen Drawing.[4] By Charles D. Maginnis. 1.00

Electric Wiring.

Vol. 13, *International Library of Technology.*[7]

Practical Lessons in Electricity.[8]

Estimating.

Estimating Frame and Brick Houses.[9] By Fred T. Hodgson. 1.00

Foundations.

Building Construction and Superintendence.[2] Part I. By F. E. Kidder. 4.00

A Treatise on Masonry Construction.[6] By Ira O. Baker. . . . 5.00

A Practical Treatise on Foundations.[6] By W. M. Patton, C E. 5.00

Furniture Designing.

Furniture Designing and Drafting.[2] By Alvan Crocker Nye. 2.00

Handbooks.

The Civil Engineers' Pocket-Book.[6] By John C. Trautwine, revised by John C. Trautwine, Jr., and John C. Trautwine, 3d. 5.00

The Mechanical Engineers' Pocket-Book.[6] By William Kent. 5.00

Carnegie Steel Company's Pocket Companion. 2.00

Cambria Steel. Published by Cambria Iron Co. 2.00

Manual of Structural Steel. Compiled by Geo. H. Blakeley and published by the Passaic Steel Co.

Steel in Construction. Published by the Pencoyd Iron Works.

Handbook. Published by the Dearborn Foundry Company, Chicago.

Book of Standards. Edited by Prof. Reid T. Stewart, and published by the National Tube Works. 1.00

The Building Trades Pocket-Book. By International Correspondence Schools, Scranton, Pa. 1.50

FIREPROOFING. Price.

The Fireproofing of Steel Buildings.[6] By J. K. Freitag. . $2.50
[A very practical and valuable work.]

HEATING AND VENTILATION.

Heating and Ventilation of Buildings.[6] By Prof. Rolla C. Carpenter. 3.00
Steam Heating for Buildings.[6] By Wm. J. Baldwin. 2.50
Vol. 32, International Library of Technology.[7]

IRON AND STEEL CONSTRUCTION—SKELETON CONSTRUCTION.

Architectural Iron and Steel[6] (many useful details). By Wm. H. Birkmire. 3.50
Compound Riveted Girders.[6] By Wm. H. Birkmire. 2.00
Skeleton Construction in Buildings.[6] By Wm. H. Birkmire. 3.00
Architectural Engineering.[6] By Joseph Kendall Freitag. 3.50
[The best discussion of the engineering problems involved in the construction of high buildings.]

PAINTING.

Rustless Coatings:[6] Corrosion and Electrolysis of Iron and Steel. By M. P. Wood. 432 pp. 4.00
The Industrial and Artistic Technology of Paints and Varnish.[6] By Alvah H. Sabin, 378 pp. 3.00

PLANNING OF CHURCHES

Churches and Chapels.[2] By F. E. Kidder. 3.00

PLANNING OF OFFICE BUILDINGS.

The Planning and Construction of High Office Buildings.[6] By William H. Birkmire. 3.50

PLANNING OF SCHOOL BUILDINGS.

Modern American School Buildings.[6] By Warren R. Briggs. 4.00
School Architecture.[10] By Edmund M. Wheelwright. 5.00

PLANNING OF THEATRES.

The Planning and Construction of American Theatres.[6] By William H. Birkmire. 3.00

Plumbing and Sanitary Engineering.

Price.

Sanitary Engineering of Buildings. Vol. I. By W. P. Gerhard. $5.00

Roof-Trusses.

(See also Steel-Mill Buildings.)

Trussed Roofs and Roof-Trusses.[2] Being the third volume of *Building Construction and Superintendence.* By F. E. Kidder. Sold separately 4.00

[A complete treatise on the subject, illustrated by many examples of roof construction in wood and steel.]

The Design of Simple Roof-Trusses in Wood and Steel.[6] By Prof. Malverd A. Howe. 2.00

Graphics for Engineers, Architects, and Builders.[6] By Prof. Chas. E. Greene.

Part I. *Roof-Trusses.* Diagrams for steady load, snow, and wind. 1.25

Part II. *Bridge Trusses* 2.50

Part III. *Arches in Wood, Iron, and Stone*. 2.50

[These books are among the best of those which show how stresses may be determined by graphical solution]

Steel-Mill Buildings.

The Design of Steel-Mill Buildings.[11] By Milo S. Ketchum. 4.00

[A practical and useful work, containing many details]

Stones for Building and Decoration.

Stones for Building and Decoration.[6] By Geo. P. Merrill. . 5.00

[The most complete work published on building stones, marbles, etc.]

Much practical information on building stones is contained in *Building Construction and Superintendence*: Part I.

Publishers.—* Published abroad. [1] Longmans, Green, & Co., N. Y. [2] Wm. T. Comstock, N. Y. [3] The American Architect Co., Boston. [4] Bates & Guild Co., Boston. [5] The Macmillan Co., N. Y. [6] John Wiley & Sons, N. Y. [7] International Text-Book Co., Scranton, Pa. [8] American School of Correspondence at Armour Inst. of Technology, Chicago. [9] David Williams Co., N. Y. [10] Rogers & Manson, Boston. [11] Engineering News Publishing Co., N. Y. [12] Published by the Author. [13] McGraw Publishing Co., N. Y. [14] D. Van Nostrand Co, N. Y.

Trade References.

Because they are somewhat out of the ordinary line of building construction and equipment, the following references are given that architects and builders may know whom to consult for information and prices. All names have been inserted without the knowledge of the parties interested or any monetary consideration. Most of the manufacturers named issue valuable publications along the line of their especial product, which will be sent gratis on application.

Acetylene Gas Generators.*
Davis Acetylene Co., Elkhart, Ind.
General Acetylene Co., New York.
Niagara Falls Acetylene Gas Generator Co., Niagara Falls, N. Y.

Antifriction Drawer Slide.
Grant Pulley & Hardware Co. (Turner Slide), 25 Warren St., N. Y.

Artificial Marble.
Artificial Marble Co., St. James Building, N. Y.
Mycenian Marble Co., 524 W. 34th St., N. Y.

Automatic Fire Apparatus, Sprinklers, etc.
General Fire Extinguisher Co., Providence, R. I.
International Sprinkler Co., Philadelphia, Pa.
Niagara Fire Extinguisher Co., Akron, O.

Bells.
Meneely Bell Co., Troy, N. Y., and 177 Broadway, N. Y. City.

Blinds, Venetian, Rolling, Sliding, etc.
Burlington Venetian Blind Co., Burlington, Vt.
Jas. G. Wilson Mfg. Co., 3 W. 29th St., N. Y.

Chain Blocks.
The Yale & Towne Mfg. Co., 9 Murray St., N. Y.

Changeable Directories, for Office Buildings.
U. S. Changeable Sign Co., 150 Nassau St., N. Y.

Chimney Brick, Perforated Radial, Manufacturers.
National Pyrogranite Co., 17 Battery Pl., N. Y.

Chimneys, Self-sustaining Steel.
The Wm. B. Pollock Co., Youngstown, O.
The Reeves Bros. Co., Alliance, O.
Walsh's Holyoke Steam Boiler Works, Holyoke, Mass.

Chimneys, Tall.—Radial Brick Systems, see also p. 1229.
Alphonse Custodis Chimney Const. Co., Bennett Bldg., N. Y.
H. R. Heinicke, 160, 5th Ave., N. Y.
Steinl Improved Chimney Const. Co., Birmingham, Ala.

Chimneys, Tall.—Of Reinforced Concrete (specialists).
Weber Steel-Concrete Chimney Co., Ashland Bl'k, Chicago, Ill.

Clocks, Electric, Programme, and Tower.
Blodgett Clock Co., 141 Franklin St., Boston.
Johnson Service Co., 240 Fourth Ave., N. Y.
The Howard Clock Co., E. Boston and New York.
Prentiss Clock Improvement Co., 304 Hudson St., N. Y.

Clothes Dryers.
Chicago Clothes Dryer Works, 346 Wabash Ave., Chicago.
Peck, Williamson Co., Cincinnati, O.
The F. M. Watkins Co., Cincinnati, O.

Conservatories and Horticultural Buildings.
Hitchings & Co., 233 Mercer St. N. Y.
Lord and Burnham Co., 1133 Broadway, N. Y.

Conveyors.
Brown Hoisting Machinery Co., Cleveland, O.
C. W. Hunt Co., West New Brighton, N. Y.
Lidgerwood Mfg. Co., New York.
Link-Belt Machinery Co., Chicago.
Robbins Conveying Belt Co., New York.

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* See The Buyers' Reference for long list of Manufacturers.

Cranes and Hoists.*—See also Derricks.
Brown Hoisting Machinery Co., Cleveland, O.
General Electric Co., Schenectady, N. Y.
Sprague Electric Co., New York.
Whiting Foundry Equipment Co., Harvey, Ill.
Yale & Towne Mfg. Co., 9 Murray St., N. Y.

Creosoting Works.
American Creosote Works, New Orleans, La.
International Creosoting and Construction Co., Galveston, Tex.

Derricks.—See also Cranes and Hoists.
American Hoist & Derrick Co., St. Paul, Minn.

Domestic Water and Light Plants.—See also Pumps.
Fairbanks, Morse & Co., Monroe St., Chicago.

Electric Blue Printing Outfits, for making blue prints by means of arc lamps.
General Electric Co., Schenectady, N. Y.
J. H. Wagenhorst & Co., Mansfield, O.

Electric Fans and Fan Motors.
General Electric Co., Schenectady, N. Y.

Elevators.*
Otis Elevator Co., principal office 17 Battery Place, N. Y.
Reedy Elevator Co., New York & Cincinnati, O.
Sprague Elevator Co., New York.
Standard Plunger Elevator Co., New York.
Warner Elevator Mfg. Co., Cincinnati, O.
Winslow Elevator and Machinery Co., Chicago.

Fibre, Hard, in Sheets. For insulating under columns, etc.
Delaware Hard Fibre Co., Wilmington, Del.

Filters.*
Albany Filter Co., New York.
Hygeia Filter Co., Detroit, Mich.
Wm. B. Scaife & Sons, Pittsburg, Pa.

Fireproof Doors and Shutters, and standard fixtures (underwriters' requirements).
Coburn Trolley Track Mfg. Co., Holyoke, Mass.

Foundry Equipment.
Whiting Foundry Equipment Co., Harvey, Ill.

Garbage Furnaces.
American Process Co., 62 William St., N. Y.
Morse-Boulger Destroyer Co., 39 Cortlandt St., N. Y.
Smith-Siemens Garb. Incin. Furnaces, 141 Broadway, N. Y.

Gasolene Engines.
De La Vergne Machine Co., E. 138th St., N. Y.
Fairbanks, Morse & Co., Monroe St., Chicago.

Hoists (Pneumatic).—See also Cranes and Derricks.
The General Pneumatic Tool Co., Montour Falls, N. Y.

Hot Air Pumping Engines.
Rider-Ericsson Mfg. Co., New York, Chicago, and Boston.

Hydraulic Rams.
N. O. Nelson Mfg. Co., St. Louis.
U. S. Wind Engine & Pump Co., Batavia, Ill.

Ice Machines.
Remington Machine Co., Wilmington, Del.
The Singer Automatic Ice Machine Co., Bridgeport, Conn.

Industrial Railways and Equipment.
Arthur Koppel, 66 Broad St., N. Y.
C. W. Hunt Co., West New Brighton, N. Y.
Wonham-Magor Engineering Co., 29 Broadway, N. Y.

Insulating Materials for Cold Storage.
Samuel Cabot, Boston.
H. W. Johns-Manville Co., 100 William St., N. Y.
Union Fibre Co., Winona, Minn.
Also manufacturers of Mineral Wool; see Buyers' Reference.

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* See The Buyers' Reference for long list of Manufacturers.

Jails and Jail Cells.
L. Screiber & Sons Co., Cincinnati, O.
The Van Dorn Iron Works Co., Cleveland, O.

Laundry Machinery.
American Laundry Machinery Co., 42 Cortlandt St., N. Y.
Chicago Clothes Dryer Works, 346 Wabash Ave., Chicago.
A. T. Hagen Co., Chicago, Ill.
Steel Roll Mangle Co., Chicago, Ill.
Troy Laundry Machinery Co., Troy, N. Y.

Library Stacks.
Art Metal Construction Co., Jamestown, N. Y.
Geo. Stykeman, 280 Broadway, N. Y.
J. B. & J. M. Cornell, 26th St. & 11th Ave., N. Y.
Library Bureau, 530 Atlantic Ave., Boston.
Snead Architectural Iron Works, Louisville, Ky.
A. B. & W. T. Westervelt, 102 Chambers St., N. Y.

Lightning Rods.
Bacon & Co., Cleveland, O.
Bajohr Lightning Rod Works, St. Louis, Mo.
Franklin Lightning Rod Works, St. Louis, Mo.
E. G. Washburn & Co., New York.

Metal Window Frames and Sash.—See p. 767.

Parquet Floors and Borders.*
The Interior Hardwood Co., Indianapolis, Ind.
S. C. Johnson & Son, Racine, Wis.

Piling, Concrete.
Raymond Concrete Pile Co., 135 Adams St., Chicago, Ill.
Simplex Concrete Piling Co., 915 Penn. Building, Philadelphia, Pa.
† Crawford Paving Co., Home Life Building, Washington, D. C.
† The Foundation Co., 35 Nassau St., N. Y.
† The Foundation Co., McCague Building, Omaha, Neb.

Piling, Sheet, Interlocking Steel.
U. S. Steel Piling Co., 135 Adams St., Chicago, Ill.
Friestedt Interlocking Channel Bar Co., 1408 Tribune Bldg., Chicago, Ill.

Pneumatic Tools.
Ingersoll-Sergeant Drill Co., The, 26 Cortlandt St., N. Y.
Philadelphia Pneumatic Tool Co., Philadelphia.
Thos. H. Dallett Co., Philadelphia, Pa.

Pumping by Compressed Air.
Ingersoll-Sergeant Drill Co., The, 26 Cortlandt St., New York
Pneumatic Engineering Co., 85 Cedar St., N. Y.

Pumps for Domestic Purposes.*
The American Works, Aurora, Ill.
The Deming Co., Salem, O.
Fairbanks, Morse & Co., Monroe St., Chicago.
The Goulds Mfg. Co., Seneca Falls, N. Y.
Rider-Ericsson Mfg. Co. (Hot-Air), New York and Chicago.

Refrigerators.—See p. 1492.

Revolving Doors.
Van Kannel Revolving Door Co., 524 E. 134th St., N. Y.

Rolling Shutters (Steel).
Columbus Steel Rolling-Shutter Co., Columbus, O.
Kinnear Mfg. Co., Columbus, O.
Rolling Steel Shutter Works, 162 W. 27th St., N. Y
Jas. G. Wilson, Mfg. Co., 3 W. 29th St., N. Y.

Roofing Tiles.*—See p. 1430.

Safes and Vaults.*
Diebold Safe and Lock Co., Canton, O.
Herring-Hall-Marvin Safe Co., Hamilton, O.
Victor Safe and Lock Co., Cincinnati, O.

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* See The Buyers' Reference for long list of Manufacturers.
† Licensed by the Simplex Concrete Piling Co.

Safety Treads.
American Mason Safety Tread Co., 40 Water St., Boston, Mass.
American Pressed Steel Co., Witherspoon Building, Philadelphia.
New York Belting and Packing Co., New York.

Sash-lifting Apparatus, for Monitor windows, greenhouses, etc.
The G. Drouvé Co., Bridgeport, Conn.
Hitchings & Co., 233 Mercer St., N. Y.
Lord & Burnham Co., 1133 Broadway, N. Y.

Sash Weights, compressed lead.
Raymond Lead Co., Lake & Clinton Sts., Chicago.

Sewerage Disposal Apparatus.
Newport Foundry and Machine Co., Newport, R. I.

Snow Guards.
Folsom Snow Guard Co., Roslindale (Boston), Mass.

Stand-pipes.
The Wm. B. Pollock Co., Youngstown, O.
The Reeves Bros. Co., Alliance, O.
Walsh's Holyoke Steam Boiler Works, Holyoke, Mass.

Sun Dials.
E. B. Meyrowitz, 104 E. 23d St., N. Y.

Swinging Hose Racks, for use in connection with stand-pipes.
H. J. M. Howard, 915 E. St., N. W., Washington, D. C.
Wirt & Knox Mfg. Co., 22 N. Fourth St., Philadelphia, Pa.

Tall Chimney Construction.—See p. 1229.

Tanks, Large Wooden.
W. E. Caldwell Co., Louisville, Ky.
Flint & Walling Mfg. Co., Kendallville, Ind.
U. S. Wind Engine & Pump Co., Batavia, Ill.

Tanks, Steel, also towers for supporting same.
Flint & Walling Mfg. Co., Kendallville, Ind.
Chicago Bridge and Iron Works, 105th & Troop Sts., Chicago.

Telephones,* for connecting portions of large buildings.
De Veau Telephone Mfg. Co., 27 Rose St., N. Y.
Electric Gas Lighting Co., 113 Purchase St., Boston.
The Simplex Interior Telephone Co., 19 E. Third St., Cincinnati.

Thermostats and Temperature Regulators.*
Davis & Roesch Temperature Controlling Co., 136 Liberty St., N. Y.
Howard Thermostat Co., Oswego, N. Y.
Johnson Electric Service Co., Milwaukee, Wis.
National Regulator Co., Chicago, Ill.

Tiles, Roofing.—See p. 1430.

Tiles, Rubber.
Goodyear Tire & Rubber Co., Akron, O.
Gutta Percha & Rubber Mfg. Co., New York.
Manhattan Rubber Mfg. Co., New York, N. Y.
Mechanical Rubber Co., Chicago, Ill.
New York Belting & Packing Co., New York.
Peerless Rubber Mfg. Co., New York, N. Y.

Travelling Cranes.—See Cranes.

Water Proofing for Brick and Stone.
Szerelmey & Co., Home Life Building, Washington, D. C.
Toch Brothers, 470 West Broadway, N. Y.

Wind Mills.
U. S. Wind Engine & Pump Co., Batavia, Ill.
Challenge Wind Mill Co., Batavia, Ill.
Flint & Walling Mfg. Co., Kendallville, Ind.

Wire Rope, Wire Rope Tramways, Transmission of Power by.
Broderick & Bascom Rope Co., St. Louis, Mo.
A. Leschen & Sons Rope Co.,† St. Louis, Mo.
John A. Roebling Sons' Co.,† Trenton, N. J.
The Trenton Iron Co.,† Trenton, N. J.

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* See The Buyers' Reference for long list of Manufacturers.
† Publish valuable pamphlets on these subjects.

GLOSSARY

OF TECHNICAL TERMS, ANCIENT AND MODERN, USED BY ARCHITECTS, BUILDERS, AND DRAUGHTSMEN.

(*Compiled by the author from various sources.*)

Aaron's-Rod.—An ornamental figure representing a rod with a serpent twined about it. It is sometimes confounded with the caduceus of Mercury. The distinction between the caduceus and the Aaron's-rod is that the former has two serpents twined in opposite directions, while the latter has but one.

Abacus.—The upper member of the capital of a column. It is sometimes square and sometimes curved, forming on the plan segments of a circle called the arch of the abacus, and is commonly decorated with a rose or other ornament in the centre, having the angles, called horns of the abacus, cut off in the direction of the radius or curve. In the Tuscan or Doric, it is a square tablet; in the Ionic, the edges are moulded; in the Corinthian, its sides are concave and frequently enriched with carving. In Gothic pillars it has a great variety of forms.

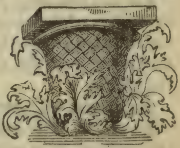
CORINTHIAN DORIC ABACUS.

Abbey.—A term for the church and other buildings used by conventual bodies presided over by an abbot or abbess, in contradistinction to cathedral, which is presided over by a bishop; and priory, the head of which was a prior or prioress.

Abutment.—That part of a pier from which the arch springs.

Abuttals.—The boundings of a piece of land on other land, street, river, etc.

Acanthus.—A plant found in the south of Europe, representations of whose leaves are employed for decorating the Corinthian and Composite capitals. The leaves of the acanthus are used on the bell of the capital, and distinguish the two rich orders from the three others.


ACANTHUS.

Acroteria.—The small pedestals placed on the extremities and apex of a pediment. They are usually without bases or plinths, and were originally intended to receive statues.

Aile, Aisle.—The wings; inward side porticos of a church; the inward lateral corridors which enclose the choir, the presbytery, and the body of the church along its sides. 2. Any one of the passages in a church or hall into which the pews or seats open.

Alcove.—The original and strict meaning of this word, which is derived from the Spanish *alcoba*, is confined to that part of a bed-chamber in which the bed stands, separated from the other parts of the room by columns or pilasters. It is now commonly used to express any large recess in a room, generally separated by an arch.

Alipterion.—In ancient Roman architecture, a room used by bathers for anointing themselves.

Almonry.—The place or chamber where alms were distributed to the poor in churches, or other ecclesiastical building. At Bishopstone Church, Wiltshire, England, it is a sort of covered porch attached to the south transept, but not communicating with the interior of the church. At Worcester Cathedral, England, the alms are said to have been distributed on stone tables, on each side, within the great porch. In large monastic establishments, as at Westminster, it seems to have been a separate building of some importance, either joining the gate-house or near it, that the establishment might be disturbed as little as possible.

Altar.—In ancient Roman architecture, a place on which offerings or sacrifices were made to the gods. In Protestant churches, the communion table is often designated as the Altar, and in Roman Catholic churches it is a square table placed at the east end of the church for the celebration of mass.

Altar of Incense.—A small table covered with plates of gold on which was placed the smoking censer in the temple at Jerusalem.

Altar-piece.—The entire decorations of an altar; a painting placed behind an altar.

Altar-screen.—The back of the altar from which the canopy was suspended, and separating the choir from the lady chapel and presbytery. The Altar-screen was generally of stone, and composed of the richest tabernacle work of niches, finials, and pedestals, supporting statues of the tutelary saints.

Alto-rilievo.—High relief—a sculpture, the figures of which project from the surface on which they are carved.

Ambo.—A raised platform, a pulpit, a reading-desk, a marble pulpit—an oblong enclosure in ancient churches, resembling in its uses and positions the modern choir.

Ambry.—A cupboard or closet, frequently found near the altar in ancient churches to hold sacred utensils.

Ambulatory.—An alley—a gallery—a cloister.

Amphiprostylos.—A Grecian temple which has a columned portico on both ends.

Amphitheatre.—A double theatre, of an elliptical form on the plan, for the exhibition of the ancient gladiatorial fights and other shows. Its arena or pit, in which those exhibitions took place, was encompassed with seats rising above each other, and the exterior had the accommodation of porticos or arcades for the public.

Amphora.—A Grecian vase with two handles, often seen on medals.

Ancones.—The consoles or ornaments cut on the key-stones of arches or on the sides of door-cases. They are sometimes made use of to support busts or other figures.

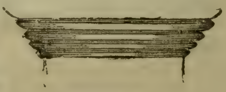
Angle-bar.—In joinery, an upright bar at the angles of polygonal windows; a mullion.

Angle-capital.—In Greek architecture, those Ionic capitals placed on the flank columns of a portico, which have one of their volutes placed horizontally at an angle of a hundred and thirty-five degrees with the plane of the frieze.

Annulated Columns.—Columns clustered together by rings or bands; much used in English architecture.

Annular Vault.—A vault rising from two parallel walls—the vault of a corridor. Same as *Barrel Vault*.

Annulet.—A small square moulding used to separate others. The fillet which separates the flutings of columns is sometimes known by this term.



ANNULET.

Anta, Antæ.—A name given to a pilaster when attached to a wall. Vitruvius calls pilasters *parastatæ* when insulated. They are not usually diminished, and in all Greek examples their capitals are different from those of the columns they accompany.

Antechamber.—An apartment preceded by a vestibule and from which is approached another room.

Antechapel.—A small chapel forming the entrance to another. There are examples at Merton College, Oxford, and at King's College, Cambridge, England, besides several others. The antechapel to the lady-chapel in cathedrals is generally called the Presbytery.

Antechoir.—The part under the rood loft, between the doors of the choir and the outer entrance of the screen, forming a sort of lobby. It is also called the Fore-choir.

Antefixa.—In classical architecture (gargoyles, in Gothic architecture), the ornaments of lions' and other heads below the eaves of a temple, through channels in which, usually by the mouth, the water is carried from the eaves. By some this term is applied to the upright ornaments above the eaves in ancient architecture, which hid the ends of the Harmi or joint tiles.

ANTEFIXA.

Apophyge.—The lowest part of the shaft of an Ionic or Corinthian column, or the highest member of its base if the column be considered as a whole. The Apophyge is the inverted cavetto or concave sweep, on the upper edge of which the diminishing shaft rests.

Apron.—A plain or moulded piece of finish below the stool of a window, put on to cover the rough edge of the plastering.

Apse.—The semicircular or polygonal termination to the chancel of a church.

Apteral.—A temple without columns on the flanks or sides.

Aqueduct.—An artificial canal for the conveyance of water, either above or under ground. The Roman aqueducts are mostly of the former construction.

Arabesque.—A building after the manner of the Arabs. Ornaments used by the same people, in which no human or animal figures appear. Arabesque is sometimes improperly used to denote a species of ornaments composed of capricious fantastics and imaginary representations of animals and foliage so much employed by the Romans in the decorations of walls and ceilings.

Arabian Architecture.—A style of architecture the rudiments of which appear to have been taken from surrounding nations, the Egyptians, Syrians, Chaldeans, and Persians. The best preserved specimens partake chiefly of the Græco-Roman, Byzantine, and Egyptian. It is supposed that they constructed many of their finest buildings from the ruins of ancient cities.

Aræostyle.—That style of building in which the columns are distant from one another from four to five diameters. Strictly speaking, the term should be limited to intercolumniation of four diameters, which is only suited to the Tuscan order.

ARABESQUE.

Aræosystylos.—That style of building in which four columns are used in the space of eight diameters and a half; the central intercolumniation being three diameters and a half, and the others on each side being only half a diameter, by which arrangement coupled columns are introduced.

Arbores.—Large bronze candelabra, in the shape of a tree, placed on the floor of ancient churches, so as to appear growing out of it.


ARCADE.

Arcade.—A range of arches, supported either on columns or on piers, and detached or attached to the wall.

Arch.—In building, a mechanical arrangement of building materials arranged in the form of a curve, which preserves a given form when resisting pressure, and enables them, supported by piers or abutments, to carry weights and resist pressure.

Arch-buttress.—Sometimes called a flying buttress; an arch springing from a buttress or pier.

Architrave.—That part of an entablature which rests upon the capital of a column, and is beneath the frieze.

Architrave Cornice.—An entablature consisting of an architrave and cornice, without the intervention of the frieze, sometimes introduced when inconvenient to give the entablature the usual height.

Architrave of a Door.—The finished work surrounding the aperture; the upper part of the lintel is called the traverse; and the sides, the jambs.

Archives.—A repository or closet for the preservation of writings or records.

Archivolt.—A collection of members forming the inner contour of an arch, or a band or frame adorned with mouldings running over the faces or the arch-stones, and bearing upon the imposts.

Area.—The superficial contents of any figure; an open space or court within a building; also, an uncovered space surrounding the foundation walls to give light to the basement.

Arena.—The plain space in the middle of the amphitheatre or other place of public resort.

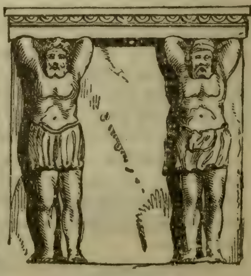
Arris.—The meeting of two surfaces producing an angle.

Arsenal.—A public storehouse for arms and ammunition.

Artificer, or **Artisan.**—A person who works with his hands, and manufactures any commodity in iron, brass, wood, etc.

Ashlar, or **Ashler.**—A facing made of squared stones, or a facing made of thin slabs, used to cover walls of brick or rubble. *Coursed ashlar* is where the stones run in level courses all around the building; *random ashlar*, where the stones are of different heights, but level beds. 2. Common freestones of small size, as they come from the quarry, are also called ashlar.

Asphaltum.—A kind of bituminous stone, principally found in the province of Neufchatel. Mixed with stone, it forms an excellent cement, incorruptible by air and impenetrable by water.


ATLANTES.

Astragal.—A small semicircular moulding, sometimes plain and sometimes ornamented.

Asymptote.—A straight line which continually approaches to a curve without touching it.

Atlases, or **Atlantes.**—Figures or half-figures of men, used instead of columns or pilasters to support an entablature; called also Telamones.

Atrium.—A court in the interior division of Roman houses.

Attached Columns.—Those which project three-fourths of their diameter from the wall.

Attic.—A low story above an entablature, or above a cornice which limits the height of the main part of an elevation. Although the term is

evidently derived from the Greek, we find nothing exactly answering to it in Greek architecture ; but it is very common in both Roman and Italian practice. What are otherwise called tholobates in St. Peter's and St. Paul's Cathedrals are frequently termed attics.

Attic Order.—A term used to denote the low pilasters employed in the decoration of an attic story.

Attributes.—In painting and sculpture, symbols given to figures and statues to indicate their office and character.

Auditory.—In ancient churches, that part of the church where the people usually stood to be instructed in the Gospel, now called the nave.

Aula.—A court or hall in ancient Roman houses.

Aviary.—A large apartment for breeding birds.

Axis.—The spindle or centre of any rotative motion. In a sphere, an imaginary line through the centre.

Back-choir.—A place behind the altar in the principal choir, in which there is, or was, a small altar standing back to back with the former.

Backing of a Rafter or Rib.—The forming of an upper or outer surface, that it may range with the edges of the ribs or rafters on either side.

Backing of a Wall.—The rough inner face of a wall ; earth deposited behind a retaining wall, etc.

Back of a Window.—That piece of wainscoting which is between the bottom of the sash frame and the floor.

Balcony.—A projection from the face of a wall, supported by columns or consoles, and usually surrounded by a balustrade.

Baldachin.—A building in the form of a canopy, supported with columns, and serving as a crown or covering to an altar.

Baluster.—A small pillar or column, supporting a rail, of various forms, used in balustrades.

Baluster Shaft.—The shaft dividing a window in Saxon architecture. At St. Albans are some of these shafts, evidently out of the old Saxon church, which have been fixed up with Norman capitals.

Balustrade.—A series of balusters connected by a rail.

Band.—A sort of flat frieze or fascia running horizontally round a tower or other parts of a building, particularly the base tables in perpendicular work, commonly used with the long shafts characteristic of the thirteenth century. It generally has a bold, projecting moulding above and below, and is carved sometimes with foliages, but in general with cusped circles, or quatrefoils, in which frequently are shields of arms.

BALDACHIN.

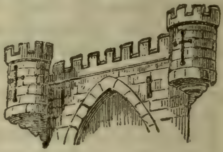
Band of a Column.—A series of annulets and hollows going round the middle of the shafts of columns, and sometimes of the entire pier. They are often beautifully carved with foliages, etc., as at Amiens. In several cathedrals there are rings of bronze apparently covering the junction of the frusta of the columns. At Worcester and Westminster they appear to have been gilt ; they are there more properly called Shaft-rings.

Baptistery.—A separate building to contain the font, for the rite of baptism. They are frequent on the Continent ; that at Rome, near St. John Lateran, and those at Florence, Pisa, Pavia, etc., are all well-known examples. The only examples in England are at Cranbrook and Canterbury ; the latter, however, is supposed to have been originally part of the treasury.

Barbican.—An outwork for the defence of a gate or drawbridge; **also, a sort** of pent-house or construction of timber to shelter warders or sentries from arrows or other missiles.

Barge Board.—See *Verge Board.*

Bartizan.—A small turret, corbelled out at the angle of a wall or tower, to protect a warder and enable him to see around him. They generally are furnished with oylets or arrow-slits.



BARTIZAN.

Basement.—The lower part of a building, usually in part below the grade of the lot or street.

Base Mouldings.—The mouldings immediately above the plinth of a wall, pillar, or pedestal.

Base of a Column.—That part which is between the shaft and the pedestal, or, if there be no pedestal, between the shaft and the plinth. The Grecian Doric had no base, and the Tuscan has only a single torus, or a plinth.

Basilica.—A term given by the Greeks and Romans to the public buildings devoted to judicial purposes.

Bas-relief.—See *Basso-rilievo.*

Basse-cour.—A court separated from the principal one, and destined for stables, etc.

Basso-rilievo, or **Bas-relief.**—The representations of figures projected from a background without being detached from it. It is divided into three parts: Alto-rilievo, when the figure projects more than one-half; Mezzo-rilievo, that in which the figure projects one-half; and Basso-rilievo, when the projection of the figure is less than one-half, as in coins.

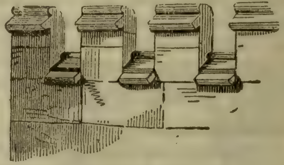
Bat.—A part of a brick.

Batten.—Small scantlings, or small strips of boards, used for various purposes. 2. Small strips put over the joints of sheathing to keep out the weather.

Batten-door.—A door made of sheathing, secured by strips of board, put crossways, and nailed with clinched nails.

Batter.—A term used by bricklayers, carpenters, etc., to signify a wall, piece of timber, or other material, which does not stand upright, but inclines from you when you stand before it; but when, on the contrary, it leans toward you, it is said to overhang.

Battlement.—A parapet with a series of notches in it, from which arrows may be shot, or other instruments of defence hurled on besiegers. The raised portions are called merlons; and the notches, embrasures or crenelles. The former were intended to cover the soldier while discharging his weapon through the latter. Their use is of great antiquity; they are found in the sculptures of Nineveh, in the tombs of Egypt, and on the famous François vase, where there is a delineation of the siege of Troy. In ecclesiastical architecture the early battlements have small shallow embrasures at some distance apart. In the Decorated period they are closer together, and deeper, and the mouldings on the top of the merlon and bottom of the embrasure are richer. During this period, and the early part of the Perpendicular, the sides or cheeks of the embrasures are perfectly square and plain. In later times the mouldings were continued round the sides, as well as at top and bottom, mitring at the angles, as over the doorway of Magdalene Col-



BATTLEMENT.

lege, Oxford, England. The battlements of the Decorated and later periods are often richly ornamented by panelling, as in the last example. In castellated work the merlons are often pierced by narrow arrow-slits. (See Oylet.) In South Italy some battlements are found strongly resembling those of old Rome and Pompeii; in the Continental ecclesiastical architecture, the parapets are very rarely embattled.

Bay.—Any division or compartment of an arcade, roof, etc. Thus each space, from pillar to pillar, in a cathedral, is called a bay, or severy.

Bay Window.—Any window projecting outward from the wall of a building, either square or polygonal on plan, and commencing from the ground. If they are carried on projecting corbels, they are called Oriel windows. Their use seems to have been confined to the later periods. In the Tudor and Elizabethan styles they are often semicircular in plan, in which case some think it more correct to call them Bow Windows.

Bazaar.—A kind of Eastern mart, of Arabic origin.

Bead.—A circular moulding. When several are joined, it is called Reeding; when flush with the surface, it is called Quirk-bead; and when raised, Cock-bead.

Beam.—A piece of timber, iron, stone, or other material, placed horizontally, or nearly so, to support a load over an opening, or from post to post.

Bearing.—The portion of a beam, truss, etc., that rests on the supports.

Bearing Wall, or **Partition.**—A wall which supports the floors and roofs in a building.

Beaufet, or **Buffet.**—A small cupboard, or cabinet, to contain china. It may either be built into a wall, or be a separate piece of furniture.

Bed.—In bricklaying and masonry, the horizontal surfaces on which the stones or bricks of walls lie in courses.

Bed of a Slate.—The lower side.

Bed Mouldings.—Those mouldings in all the orders between the corona and frieze.

Belfry.—Properly speaking, a detached tower or campanile containing bells, as at Evesham, England, but more generally applied to the ringing-room or loft of the tower of a church. See *Tower*.

Bell-cot, Bell-gable, or **Bell-turret.**—The place where one or more bells are hung in chapels, or small churches which have no towers. Bell-cots are sometimes double, as at Northborough and Coxwell, England; a very common form in France and Switzerland admits of three bells. In these countries, also, they are frequently of wood, and attached to the ridge. Those which stand on the gable, dividing the nave from the chancel, are generally called Sanctus Bells. A very curious and, it is believed, unique example at Cleves Abbey, England, juts out from the wall. In later times bell-turrets were much ornamented; these are often called Flèches.

Bell of a Capital.—In Gothic work, immediately above the necking is a deep, hollow curve; this is called the bell of a capital. It is often enriched with foliages. It is also applied to the body of the Corinthian and Composite capitals.

Belt.—A course of stones or brick projecting from a brick or stone wall, generally placed in a line with the sills of the windows; it is either moulded, fluted, plane, or enriched with patras at regular intervals. Sometimes called Stone String.

Belvedere, or **Look-out.**—A turret or lantern raised above the roof of an observatory for the purpose of enjoying a fine prospect.

Bema.—The semicircular recess, or hexedra, in the basilica, where the judges sat, and where in after-times the altar was placed. It generally is roofed with a half-dome or concha. The seats of the priests were against the wall, looking

into the body of the church, that of the bishop being in the centre. The bema is generally ascended by steps, and railed off by cancelli.

Bench Table.—The stone seat which runs round the walls of large churches, and sometimes round the piers; it very generally is placed in the porches.

Bevel.—An instrument for taking angles. One side of a solid body is said to be bevelled with respect to another, when the angle contained between those two sides is greater or less than a right angle.

Bezantee.—A name given to an ornamented moulding much used in the Norman period, resembling bezants, coins struck in Byzantium.

Billet.—A species of ornamented moulding much used in Norman, and sometimes in Early English work, like short pieces of stick cut off and arranged alternately.

Blocking, or **Blocking-course.**—In masonry, a course of stones placed on the top of a cornice crowning the walls.

Bond.—In bricklaying and masonry, that connection between bricks or stones formed by lapping them upon one another in carrying up the work, so as to form an inseparable mass of building, by preventing the vertical joints falling over each other. In brickwork there are several kinds of bond. In common brick walls in every sixth or seventh course the bricks are laid crossways of the wall, called Headers. In face work, the back of the face brick are clipped so as to get in a diagonal course of headers behind. In Old English bond, every alternate course is a header course. In Flemish bond, a header and stretcher alternate in each course.

Bond-stones.—Stones running through the thickness of the wall at right angles to its face, in order to bind it together.

Bond-timbers.—Timbers placed in a horizontal direction in the walls of a brick building in tiers, and to which the battens, laths, etc., are secured. In rubble work, walls are better plugged for this purpose.

Border.—Useful ornamental pieces around the edge of anything.

Boss.—An ornament, generally carved, forming the key-stone at the intersection of the ribs of a groined vault. Early Norman vaults have no bosses. The carving is generally foliage, and resembles that of the period in capitals, etc. Sometimes they have human heads, as at Notre Dame at Paris, and sometimes grotesque figures. In Later Gothic vaulting there are bosses at every intersection.

Boutell.—The mediæval term for a round moulding, or torus. When it follows a curve, as round a bench end, it is called a Roving Boutell.

Bow.—Any projecting part of a building in the form of an arc of a circle. A bow, however, is sometimes polygonal.

Bow Window.—A window placed in the bow of a building.

Brace.—In carpentry, an inclined piece of timber, used in trussed partitions, or in framed roofs, in order to form a triangle, and thereby stiffen the framing. When a brace is used by way of support to a rafter, it is called a strut. Braces in partitions and span-roofs are, or always should be, disposed in pairs, and introduced in opposite directions.

Brace Mould.—[{] Two ressaunts or ogees united together like a brace in printing, sometimes with a small bead between them.

Bracket.—A projecting ornament carrying a cornice. Those which support vaulting shafts or cross springers of a roof are more generally called Corbels.

Break.—Any projection from the general surface of a building.

Breaking Joint.—The arrangement of stones or bricks so as not to allow two joints to come immediately over each other. See *Bond*.

Breast of a Window.—The masonry forming the back of the recess and the parapet under the window-sill.

Bressummer.—A lintel, beam, or iron tie, intended to carry an external wall and itself supported by piers or posts ; used principally over shop windows. This term is now seldom used, the word *beam*, or *girder*, taking its place.

Bridging.—A method of stiffening floor joist and partition studs, by cutting pieces in between. Cross bridging of floor joist is illustrated in cut.

CROSS-BRIDGING.

Bulwark.—In ancient fortification, nearly the same as Bastion in modern.

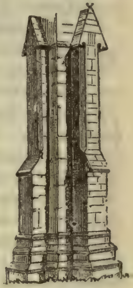
Burse, or **Bourse.**—A public edifice for the assembly of merchant traders ; an exchange.

Bust.—In sculpture, that portion of the human figure which comprises the head, neck, and shoulders.

Buttery.—A store-room for provisions.

Butt-joint.—Where the ends of two pieces of timber or moulding butt together.

Buttress.—Masonry projecting from a wall, and intended to strengthen the same against the thrust of a roof or vault. Buttresses are no doubt derived from the classic pilasters which serve to strengthen walls where there is a pressure of a girder or roof-timber. In very early work they have little projection, and, in fact, are "strippilasters." In Norman work they are wider, with very little projection, and generally stop under a cornice or corbel table. Early English buttresses project considerably, sometimes with deep sloping weatherings in several stages, and sometimes with gabled heads. Sometimes they are chamfered, and sometimes the angles have jamb shafts. At Wells and Salisbury, England, they are richly ornamented with canopies and statues. In the Decorated period they became richly panelled in stages, and often finish with niches and statues and elegantly carved and crocketed gabelts, as at York, England. In the Perpendicular period the weatherings became waved, and they frequently terminate with niches and pinnacles.

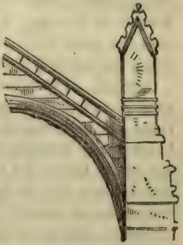


BUTTRESS.

Buttress, Flying.—A detached buttress or pier of masonry at some distance from a wall, and connected therewith by an arch or portion of an arch, so as to discharge the thrust of a roof or vault on some strong point.

Buttress Shafts.—Slender columns at the angle of buttresses, chiefly used in the Early English period.

Byzantine Architecture.—A style developed in the Byzantine Empire. The capitals of the pillars are of endless variety and full of invention ; some are founded on the Greek Corinthian, some resemble the Norman and the Lombard style, and so varied that no two sides of the same capital are alike. They are comprised under the style Romanesque, which comprehends the round-arch style. Byzantine architecture reached its height in the Church of St. Sophia at Constantinople.

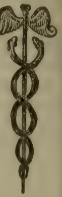


FLYING BUTTRESS.

Cabinet.—A highly ornamented kind of buffet or chest of drawers set apart for the preservation of things of value.

Cabling.—The flutes of columns are said to be cabled when they are partly occupied by solid convex masses, or appear to be refilled with cylinders after they had been formed.

Caduceus.—Mercury's rod, a wand entwined by two serpents and surmounted by two wings. The rod represents power ; the serpents, wisdom ; and the wings, diligence and activity.



CADUCEUS.

Caisson.—A panel sunk below the surface in flat or vaulted ceilings. See *Cassoon.*

Caisson.—In bridge building, a chest or vessel in which the piers of a bridge are built, gradually sinking as the work advances till its bottom comes in contact with the bed of the river, and then the sides are disengaged, being so constructed as to allow of their being thus detached without injury to its floor or bottom.

Caliber, or **Caliper.**—The diameter of any round body ; the width of the mouth of a piece of ordnance.

Camber.—In carpentry, the convexity of a beam upon the surface, in order to prevent its becoming concave by its own weight, or by the burden it may have to sustain.

Campanile.—A name given in Italy to the bell-tower of a town-hall or church In that country this is almost always detached from the latter.

Candelabrum.—Stand or support on which the ancients placed their lamps. Candelabra were made in a variety of shapes and with much taste and elegance. The term is also used to denote a tall ornamental candlestick with several arms, or a bracket with arms for candles.

Canopy.—The upper part or cover of a niche, or the projection or ornament over an altar, seat, or tomb. The word is supposed to be derived from conopæum, the gauze covering over a bed to keep off the gnats ; a mosquito curtain. Early English canopies are generally simple, with trefoiled or cinque-foiled heads ; but in the later styles they are very rich, and divided into compartments with pendants, knots, pinnacles, etc. The triangular arrangement over an Early English and Decorated doorway is often called a canopy. The triangular canopies in the North of Italy are peculiar. Those in England are generally part of the arrangement of the arch mouldings of the door, and form, as it were, the hood-moulds to them, as at York. The former are above and independent of the door mouldings, and frequently support an arch with a tympanum, above which is a triangular canopy, as in the Duomo at Florence. Sometimes the canopy and arch project from the wall, and are carried on small jamb shafts, as at San Pietro Martiro at Verona. Canopies are often used over windows, as at York Minster over the great west window, and lower ties in the towers. These are triangular, while the upper windows in the towers have ogee canopies.

Capital.—The upper part of a column, pilaster, pier, etc. Capitals have been used in every style down to the present time. That mostly used by the Egyptians was bell-shaped, with or without ornaments. The Persians used the double-headed bell, forming a kind of bracket capital. The Assyrians apparently made use of the Ionic and Corinthian, which were developed by the Greeks, Romans, and Italians into their present well-known forms. The Doric was apparently an invention or adaptation by the Greeks, and was altered by the Romans and Italians. But in all these examples, both ancient and modern, the capitals of an order are all of the same form throughout the same building, so that if one be seen the form of all the others is known. The Romanesque architects altered all this, and in the carving of their capitals often introduced such figures and emblems as helped to tell the story of their building. Another form was introduced by them in the curtain capital, rude at first, but afterward highly decorated. It evidently took its origin from the cutting off of the lower angles of a square block, and then rounding them off. The process may be distinctly seen, in its several stages, in Mayence Cathedral. But this form of capital was more

fully developed by the Normans, with whom it became a marked feature. In the early English capitals a peculiar flower of three or more lobes was used spreading from the necking upward in most graceful forms. In Decorated and Perpendicular styles this was abandoned in favor of more realistic forms of crumpled leaves, enclosing the bell like a wreath. In each style bold abacus mouldings were always used, whether with or without foliage.

Caravansary.—A huge, square building, or inn, in the East, for the reception of travellers and lodging of caravans.

Carriage.—The timber or iron joist which supports the steps of a wooden stair.

Carton, or **Cartoon.**—A design made on strong paper, to be transferred on the fresh plaster wall to be afterward painted in fresco; also, a colored design for working in mosaic tapestry.

Cartouche.—An ornament which like an escutcheon, a shield or an oval or oblong panel has the central part plain, and usually slightly convex, to receive an inscription, armorial bearings, or an ornamental or significant piece of painting or sculpture. Frequently used in French Renaissance and Modern Architecture.

Caryatides.—Human female figures used as piers, columns, or supports. *Caryatic* is applied to the human figure generally, when used in the manner of caryatides.

CARYATID.

Cased.—Covered with other materials, generally of a better quality.

Casement.—A glass frame which is made to open by turning on hinges affixed to its vertical edges.

Cassoon, or **Caisson.**—A deep panel or coffer in a soffit or ceiling. This term is sometimes written in the French form, *caisson;* sometimes derived directly from the Italian *cassone*, the augmentative of *cassa*, a chest or coffer.

Cast.—A term used in sculpture for the impression of any figure taken in plaster of Paris, wax, or other substances.

Catacombs.—Subterranean places for burying the dead. Those of Egypt, and near Rome, are believed to be the most important.

Catafalco.—An ornamental scaffold used in funeral solemnities.

Cathedral.—The principal church, where the bishop has his seat as diocesan.

Cauliculus.—The inner scroll of the Corinthian capital. It is not uncommon, however, to apply this term to the larger scrolls or volutes also.

Causeway.—A raised or paved way.

Cavetto.—A concave ornamental moulding, opposed in effect to the ovolo—the quadrant of a circle.

Ceiling.—That covering of a room which hides the joists of the floor above, or the rafters of the roof. Most European churches have either open roofs, or are groined in stone. At Peterborough and St. Albans, England, there are very old flat ceilings of boards curiously painted. In later times the boarded ceilings, and, in fact, some of those of plaster, have moulded ribs, locked with bosses at the intersection, and are sometimes elaborately carved. In many English churches there are ceilings formed of oak ribs, filled in at the spandrels with narrow, thin pieces of board, in exact imitation of stone groining. In the Elizabethan and subsequent periods the ceilings are enriched with most elaborate ornaments in stucco. 2. Matched and beaded boards, planed and smoothed, used for wainscoting. In the New England States it is called sheathing.

Cenotaph.—An honorary tomb or monument, distinguished from monuments in being empty, the individual it is to memorialize having received interment elsewhere.

Centaur.—A poetical imaginary being of heathen mythology, half-man and half-horse.

Centring.—In building, the frames on which an arch is turned.

Chamfer, Champfer, or **Chaumfer.**—When the edge or arris of any work is cut-off at an angle of 45° in a small degree, it is said to be chamfered; if to a large scale, it is said to be a canted corner. The chamfer is much used in mediæval work, and is sometimes plain, sometimes hollowed out, and sometimes moulded.

Chamfer Stop.—Chamfers sometimes simply run into the arris by a plane face; more commonly they are first stopped by some ornament, as by a bead; they are sometimes terminated by trefoils, or cinque-foils, double or single, and in general form very pleasing features in mediæval architecture.

Chancel.—A place separated from the rest of a church by a screen. The word is now generally used to signify the portion of an Episcopal or Catholic church containing the altar and communion table.

Chantry.—A small chapel, generally built out from a church. They generally contain a founder's tomb, and are often endowed places where masses might be said for his soul. The officiator, or mass priest, being often unconnected with the parochial clergy; the chantry has generally an entrance from the outside.

Chapel.—A small, detached building used as a substitute for a church in a large parish; an apartment in any large building, a palace, a nobleman's house, a hospital or prison, used for public worship; or an attached building running out of and forming part of a large church, generally dedicated to different saints, each having its own altar, piscina, etc., and screened off from the body of the building.

Chapter House.—The chamber in which the chapter or heads of the monastic bodies assembled to transact business. They are of various forms; some are oblong apartments, some octagonal, and some circular.

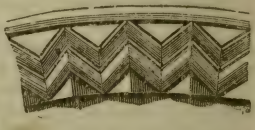
Chaptrel.—In Gothic architecture, the capital of a pier or column which receives an arch.

CHAPTREL

Charnel House.—A place for depositing the bones which might be thrown up in digging graves. Sometimes it was a portion of the crypt; sometimes it was a separate building in the church-yard; sometimes chantry chapels were attached to these buildings. M. Viollet-le-Duc has given two very curious examples of *ossuaires*—one from Fleurance, the other from Faouet.

Cherub—Gothic.—A representation of an infant's head joined to two wings, used in the churches on key-stones of arches and corbels.

Chevron—Gothic.—An ornament turning this and that way, like a zigzag, or letter Z.



CHEVRON.

Chiaro-oscuro.—The effects of light and shade in a picture.

Choir.—That part of a church or monastery where the breviary service, or "horæ," is chanted.

Church.—A building for the performance of public worship. The first churches were built on the plan of the ancient basilicæ, and afterward on the plan of a cross: a church is said to be in Greek cross when the length of the transverse is equal to that of the nave; in Latin cross, when the nave is longer than the transverse part; in rotundo, when it is a perfect circle; simple, when it has only a nave and choir; with aisles, when it has a row of porticos in form of vaulted galleries, with chapels in its circumference.

Ciborium.—A tabernacle or vaulted canopy supported on shafts standing over the high altar.

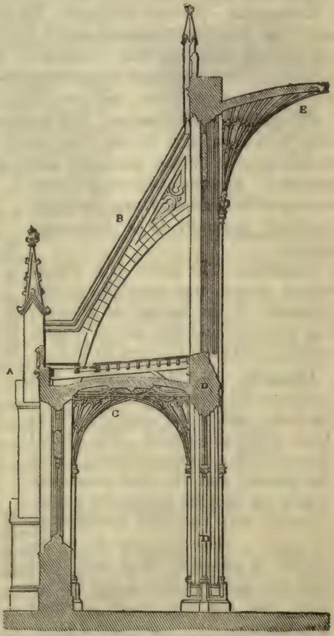
Cincture.—A ring, list, or fillet at the top and bottom of a column, serving to divide the shaft of the column from its capital and base.

CINQUE-FOIL.

Cinque-foil.—A sinking or perforation, like a flower, of five points or leaves, as a quatre-foil is of four. The points are sometimes in a circle, and sometimes form the cusping of a head.

Civic Crown.—A garland of oak-leaves and acorns, given as honorary distinction among the Romans to such as had preserved the life of a fellow-citizen.

Clere-story, Clear-story.—When the middle of the nave of a church rises above the aisles and is pierced with windows, the upper story is thus called. Sometimes these windows are very small, being mere quatre-foils, or spherical triangles. In large buildings, however, they are important objects, both for beauty and utility. The window of the clere-stories of Norman work, even in large churches, are of less importance than in the later styles. In Early English they became larger; and in the Decorated they are more important still, being lengthened as the triforium diminishes. In Perpendicular work the latter often disappears altogether, and in many later churches the clere-stories are close ranges of windows. The word *clere-story* is also used to denote a similar method of lighting other buildings besides churches, especially factories, depots, sheds, etc.



Bath Abbey.

FLYING BUTTRESS AND CLERE-STORY.

A, buttress with pinnacle; B, flying buttress supporting clere-story; C, vaulted roof of aisle; D D, pier dividing nave from aisle; E, vaulted roof of nave.

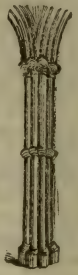
Cloister.—An enclosed square, like the atrium of a Roman house, with a walk or ambulatory around, sheltered by a roof, generally groined, and by tracery windows, which were more or less glazed.

Close.—The precinct of a cathedral or abbey. Sometimes the walls are traceable, but now generally the boundary is only known by tradition.

Close String, or Box String.—A method of finishing the outer edge of stairs, by building up a sort of curb string on which the balusters set, and the treads and risers stop against it.

Clustered.—In architecture, the coalition of several members which penetrate each other.

Clustered Column.—Several slender pillars attached to each other so as to form one. The term is used in Roman architecture to denote two or four columns which appear to intersect each other at the angle of a building to answer at each return.



CLUSTERED COLUMN.

Coat.—A thickness or covering of paint, plaster, or other work, done at one time. The first coat of plastering is called the scratch coat, the second coat (when there are three coats) is called the brown coat, and the last coat is variously known as the slipped coat, skim coat, or white coat. It varies in composition in different localities.

Coffer.—A deep panel in a ceiling.

Coffer Dam.—A frame used in the building of a bridge in deep water, similar to a caisson.

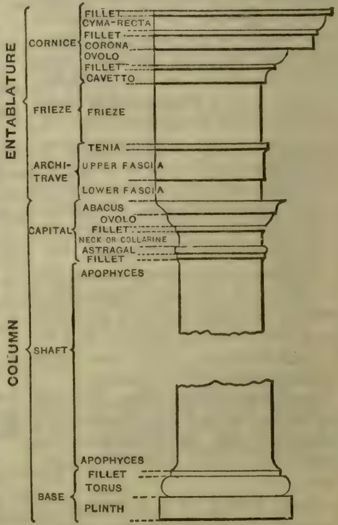
Collar Beam.—A beam above the lower ends of the rafters, and spiked to them.

Colonnade.—A row of columns. The colonnade is termed, according to the number of columns which support the entablature: Tetrastyle, when there are four; hexastyle, when six; octostyle, when eight, etc. When in front of a building they are termed porticos; when surrounding a building, peristyle; and when double or more, polystyle.

Colosseum, or **Coliseum.**—The immense amphitheatre built at Rome by Flavius Vespasian, A.D. 72, after his return from his victories over the Jews. It would contain ninety thousand persons sitting, and twenty thousand more standing. The name is now employed to denote an unusually large audience building, generally of a temporary nature.

Colossus.—The name of a brazen statue which was erected at the entrance of the harbor at Rhodes, one hundred and five feet in height. Vessels could sail between its legs.

Column.—A round pillar. The parts are the base, on which it rests; its body, called the shaft; and the head, called the capital. The capital finishes with a horizontal table, called the abacus, and the base commonly stands on another, called the plinth. Columns may be either insulated or attached. They are said to be attached or engaged when they form part of a wall, projecting one-half or more, but not the whole, of their substance.



SECTION OF COLUMN AND ENTABLATURE.
(Divided according to the Tuscan Order.)

Common.—A line, angle, surface, etc., which belongs equally to several objects. Common centring is a centring without trusses, having a tie beam at bottom. Common joists are the beams in naked flooring to which the joists are fixed. Common rafters in a roof are those to which the laths are attached.

Composite Arch.—Is the pointed or lancet arch.

Composite Order.—The most elaborate of the orders of classical architecture.

Concrete.—A mass composed of broken stone, sand, and hydraulic cement, which makes a sort of artificial stone, much used for foundations; a finer variety is sometimes used in blocks for building houses.

Conduit.—A long narrow passage between two walls or underground for secret communication between different apartments; also, a canal or pipe for the conveyance of water.

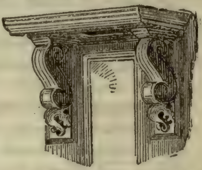
Confessional.—The seat where a priest or confessor sits to hear confessions.

Congé.—Another name for the echinus or quarter round.

Conservatory.—A building for the protection and rearing of tender plants, often attached to a house as an apartment. Also, a public place of instruction, designed to preserve and perfect the knowledge of some branch of learning or the fine arts; as, a *conservatory* of music.

Consistory.—The judicial hall of the College of Cardinals at Rome.

Consol, or Console.—A bracket or truss, generally with scrolls or volutes at the two ends, of unequal size and contrasted, but connected by a flowing line from the back of the upper one to the inner convolving face of the lower.



CONSOLES.

Coping.—The capping or covering of a wall. This is of stone, weathered to throw off the wet. In Norman times, as far as can be judged from the little there is left, it was generally plain and flat, and projected over the wall with a throating to form a drip. Afterward it assumed a torus or bowtell at the top, and became deeper, and in the Decorated period there were generally several sets-off. The copings in the Perpendicular period assumed something of the wavy section of the buttress caps, and mitred round the sides of the embrasure, as well as the top and bottom.

Corbel.—The name, in mediæval architecture, for a piece of stone jutting out of a wall to carry any superincumbent weight. A piece of timber projecting in the same way was called a tassel or a bragger. Thus, the carved ornaments from which the vaulting shafts spring at Lincoln are corbels. Norman corbels are generally plain. In the Early English period they are sometimes elaborately carved. They sometimes end with a point, apparently growing into the wall, or forming a knot, and often are supported by angles and other figures. In the later periods the foliage or ornaments resemble those in the capitals. In modern architecture, a short piece of stone or wood projecting from a wall to form a support, generally ornamented.

Corbel Out.—To build out one or more courses of brick or stone from the face of a wall, to form a support for timbers.

Corbel Table.—A projecting cornice or parapet, supported by a range of corbels a short distance apart, which carry a moulding, above which is a plain piece of projecting wall forming a parapet, and covered by a coping. Sometimes small arches are thrown across from corbel to corbel, to carry the projection.

Cornice.—The projection at the top of a wall finished by a blocking-course, common in classic architecture. In Norman times, the wall finished with a corbel table, which carried a portion of plain projecting work, which was finished by a coping, and the whole formed a parapet. In Early English times the parapet was much the same, but the work was executed in a much better way, especially the small arches connecting the corbels. In the Decorated period the corbel table was nearly abandoned, and a large hollow, with one or two subordinate mouldings, substituted; this is sometimes filled with the ball flowers, and sometimes with running foliages. In the Perpendicular style the parapet frequently

did not project beyond the wall-line below ; the moulding then became a string (though often improperly called a cornice), and was ornamented by a quatre-foil, or small rosettes, set at equal intervals immediately under the battlements. In many French examples the moulded string is very bold, and enriched with foliage ornaments.

Corona.—The brow of the cornice which projects over the bed mouldings to throw off the water.

Corridor.—A long gallery or passage in a mansion connecting various apartments and running round a quadrangle. Any long passage-way in a building.

Countersink.—To make a cavity for the reception of a plate of iron, or the head of a screw or bolt, so that it shall not project beyond the face of the work.

Coupled Columns.—Columns arranged in pairs.

Course.—A continued layer of bricks or stones in buildings ; the term is also applicable to slates, shingles, etc.

Court.—An open area behind a house, or in the centre of a building and the wings. Courts admit of the most elegant ornamentations, such as arcades, etc.

Cove—Coving.—The moulding called the cavetto, or the scotia inverted, on a large scale, and not as a mere moulding in the composition of a cornice, is called a cove or a coving.

Cove-bracketing.—The wooden skeleton mould or framing of a cove, applied chiefly to the bracketing of a cove ceiling.

Cove Ceiling.—A ceiling springing from the walls with a curve.

Coved and Flat Ceiling.—A ceiling in which the section is the quadrant of a circle, rising from the walls and intersecting in a flat surface.

Cradling.—Timber work for sustaining the lath and plaster of vaulted ceilings.

Cresting.—An ornamental finish in the wall or ridge of a building, which is common on the Continent of Europe. An example occurs at Exeter Cathedral, the ridge of which is ornamented with a range of small fleurs-de-lis in lead.

Crocket.—An ornament running up the sides of gablets, hood-moulds, pinnacles, spires: generally, a winding stem like a creeping plant, with flowers or leaves projecting at intervals, and terminating in a finial.

CROCKET.

Cross.—This religious symbol is almost always placed on the ends of gables, the summit of spires, and other conspicuous places of old churches. In early times it was generally very plain, often a simple cross in a circle. Sometimes they take the form of a light cross, crosslet, or a cross in a square. In the Decorated and later styles they became richly floriated, and assumed an endless variety of forms. Of memorial crosses the finest examples are the Eleanor crosses, erected by Edward I. Of these a few yet remain, one of which has recently been reërected at Charing Cross. Preaching crosses were often set up by the wayside as stations for preaching ; the most noted is that in front of St. Paul's, England. The finest remaining sepulchral crosses are the old elaborately carved examples found in Ireland.

Cross-aisle.—An old name for a transept.

Cross-springer.—The transverse ribs of a vault.

Cross-vaulting.—A common name given to groins and cylindrical vaults.

Crown.—In architecture the uppermost member of the cornice: called also Corona and Larmier.

Crypt.—A vaulted apartment of greater or less size, usually under the choir.

Cupola.—A small room, either circular or polygonal, standing on the top of a dome. By some it is called a Lantern.

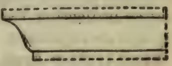
Curb Roof, or **Mansard Roof**.—A roof formed of four contiguous planes, each two having an external inclination

Curtail Step.—The first step in a stair. which is generally finished in the form of a scroll.

Cusp.—The point where the foliations of tracery intersect. The earliest example in England of a plain cusp is probably that at Pythagoras School at Cambridge ; of an ornamental cusp at Ely Cathedral. where a small roll, with a rosette at the end, is formed at the termination of a cusp. In the later styles the terminations of the cusps were more richly decorated ; they also sometimes terminate not only in leaves or foliages, but in rosettes, heads, and other fanciful ornaments.

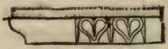
Cyclostyle.—A structure composed of a circular range of columns without a core is cyclostylar ; with a core, the range would be a peristyle. This is the species of edifice called by Vitruvius monopteral.

Cyma.—The name of a moulding of very frequent use. It is a simple, waved line, concave at one end and convex at the other. like an Italic *f*. When the concave part is uppermost it is called a cyma recta , but if the convexity appear above, and the concavity below it is then a cyma reversa.



CYMA RECTA.

Cymatium.—When the crowning moulding of an entablature is of the cyma form, it is termed the Cymatium.



CYMA REVERSA.

Cyrtostyle.—A circular projecting portico. Such are those of the transept entrances to St Paul's Cathedral. London

Dado, or **Die**.—The vertical face of an insulated pedestal between the base and cornice, or surbase. It is extended also to the similar part of all stereobates which are arranged like pedestals in Roman and Italian architecture

Dais.—A part of the floor at the end of a mediæval hall, raised a step above the rest of the floor. On this the lord of the mansion dined with his friends at the great table, apart from the retainers and servants. In mediæval halls there was generally a deep recessed bay window at one or at each end of the dais. supposed to be for retirement, or greater privacy than the open hall could afford. In France the word is understood as a canopy or hanging over a seat ; probably the name was given from the fact that the seats of great men were then surmounted by such an ornament

Darby.—A flat tool used by plasterers in working, especially on ceilings It is generally about seven inches wide and forty-two inches long with two handles on the back.

Decastyle.—A portico of ten columns in front

Decorated Style.—The second stage of the Pointed or Gothic style of architecture, considered the most complete and perfect development of Gothic architecture, the best examples of which are found in England

Demi-metope.—The half of a metope, which is found at the retiring or projecting angles of a Doric frieze.

Dentil.—The cogged or toothed member common in the bed-mould of a Corinthian entablature, is said to be dentilled. and each cog or tooth is called a dentil.

Depressed Arches, or **Drop Arches**.—Those of less pitch than the equilateral.

Design.—The plans, elevations, sections, and whatever other drawings may be necessary for an edifice, exhibit the design, the term plan having a restricted application to a technical portion of the design

Detail.—As used by architects, detail means the smaller parts into which a

composition may be divided. It is applied generally to mouldings and other enrichments, and again to their minutiæ.

Diameter.—The line in a circle passing through its centre, or thickest part, which gives the measure proportioning the intercolumniation in some of the orders.

Diameters.—The diameters of the lower and upper ends of the shaft of a column are called its inferior and superior diameters, respectively ; the former is the greatest, the latter the least diameter of the shaft.

Diaper.—A method of decorating a wall, panel, stained glass, or any plain surface, by covering it with a continuous design of flowers, rosettes, etc., either in squares or lozenges, or some geometrical form resembling the pattern of a diapered table-cloth, from which, in fact, the name is supposed by some to have been derived.

Diastyle.—A spacious intercolumniation, to which three diameters are assigned.

Dipteros.—A double-winged temple. The Greeks are said to have constructed temples with two ranges of columns all around, which were called dipteroi. A portico projecting two columns and their interspaces is of dipteral or pseudodipteral arrangement.

Discharging Arch.—An arch over the opening of a door or window, to discharge or relieve the superincumbent weight from pressing on the lintel.

Distemper.—Term applied to painting with colors mixed with size or other glutinous substance. All the cartoons of the ancients, previous to the year 1410, are said to be done in distemper.

Distyle.—A portico of two columns. This is not generally applied to the mere porch with two columns, but to describe a portico with two columns *in antis*.

Ditriglyph.—An intercolumniation in the Doric order, of two triglyphs.

Dodecastyle.—A portico of twelve columns in front. The lower one of the west front of St. Paul's Cathedral, London, is of twelve columns, but they are coupled, making the arrangement pseudo-dodecastyle. The Chamber of Deputies in Paris has a true dodecastyle.

Dog-tooth.—A favorite enrichment used from the latter part of the Norman period to the early part of the Decorated. It is in the form of a four leaved flower, the centre of which projects, and probably was named from its resemblance to the dog-toothed violet.

Dome.—A cupola or inverted cup on a building. The application of this term to its generally received purpose is from the Italian custom of calling an archiepiscopal church, by way of eminence, Il Duomo, the temple ; for to one of that rank, the Cathedral of Florence, the cupola was first applied in modern practice. The Italians themselves never call a cupola a dome ; it is on this side of the Alps the application has arisen, from the circumstance, it would appear, that the Italians use the term with reference to those structures whose most distinguishing feature is the cupola, tholus, or (as we now call it) dome.

Domestic Architecture.—That branch which relates to private buildings.

Donjon.—The principal tower of a castle, generally containing the prison.

Door Frame.—The surrounding case into and out of which the door shuts and opens. It consists of two upright pieces, called jambs, and a head, generally fixed together by mortices and tenons, and wrought, rebated, and beaded.

Doric Order.—The oldest of the three orders of Grecian architecture.

Dormer Window.—A window belonging to a room in a roof, which consequently projects from it with a valley gutter on each side. They are said not to be earlier than the fourteenth century. In Germany there are often several rows

of dormers, one above the other. In Italian Gothic they are very rare; in fact, the former have an unusually steep roof, while in the latter country, where the Italian tile is used, the roofs are rather flat.

Dormitory.—A room, suite of rooms, or building used to sleep in. The name was first applied to the place where the monks slept at night. It was sometimes one long room like a barrack, and sometimes divided into a succession of small chambers or cells. The dormitory was generally on the first floor, and connected with the church, so that it was not necessary to go out-of doors to attend the nocturnal services. In the large houses of the Perpendicular period, and also in some of the Elizabethan, the entire upper story in the roof formed one large apartment, said to have been a place for exercise in wet weather, and also for a dormitory for the retainers of the household, or those of visitors.

Double Vault.—Formed by a duplicate wall; wine cellars are sometimes so formed.

Dovetailing.—In carpentry and joinery, the method of fastening boards or other timbers together, by letting one piece into another in the form of the expanded tail of a dove.

Dowel.—1. A pin let into two pieces of wood or stone, where they are joined together. 2. A piece of wood driven into a wall so that other pieces may be nailed to it. This is also called plugging.

Draw-bridge.—A bridge made to draw up or let down, much used in fortified places. In navigable rivers, the arch over the deepest channel is made to draw or revolve, in order to let the masts of ships pass through.

Drawing-room.—A room appropriated for the reception of company; a room to which company withdraws from the dining room.

Dresser.—A cupboard or set of shelves to receive dishes and cooking utensils.

Dressing.—Is the operation of squaring and smoothing stones for building; also applied to smoothing lumber.

Dressing-room.—An apartment appropriated for dressing the person.

Drip.—A name given to the member of a cornice which has a projection beyond the other parts for throwing off water by small portions, drop by drop. It is also called Larmier.

Drip-stone.—The label moulding which serves on a canopy for an opening, and to throw off the rain. It is also called Weather Moulding.

Drop-scene.—A curtain suspended by pulleys, which descends or drops in front of the stage in a theatre.

Drum.—The upright part of a cupola over a dome; also, the solid part or vase of the Corinthian and Composite capitals.

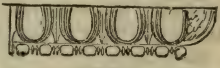
Dry-rot.—A rapid decay of timber, by which its substance is converted into a dry powder, which issues from minute cavities resembling the borings of worms.

Dungeon.—The prison in a castle keep, so called because the Norman name for the latter is donjon, and the dungeons, or prisons, are generally in its lowest story.

Dwarf Wall.—The walls enclosing courts above which are railings of iron; low walls, in general, receive this name.

Eaves.—In slating and shingling, the margin or lower part of the slating hanging over the wall, to throw the water off from the masonry or brickwork.

Echinus.—A moulding of eccentric curve, generally cut (when it is carved) into the forms of eggs and anchors alternating, whence the moulding is called by the name of the more conspicuous. It is the same as Ovolo.



ECHINUS.

Edifice.—Is synonymous with the terms building, fabric, erection, but is more strictly applicable to architecture distinguished for size, dignity, and grandeur.

Efflorescence.—In architecture, the formation of a whitish loose powder, or crust, on the surface of stone or brick walls.

Egyptian Architecture.—The earliest civilization and cultivation of the arts was in Upper Egypt. The most remarkable and most ancient monuments of the Egyptians, with the exception of the pyramids, are nearly all included in Upper Egypt. The buildings of Egypt are characterized by solidity and massiveness of construction, originality of conception, and boldness of form. The walls, the pillars, and the most sacred places of their religious buildings were ornamented with hieroglyphics and symbolical figures, while the ceilings of the porticos exhibited zodiacs and celestial planispheres. The temples of Egypt were generally without roofs, and, consequently, the interior colonnades had no pediments, supporting merely an entablature, composed of only architrave, frieze, and cornice, formed of immense blocks united without cement and ornamented with hieroglyphics.

Element.—The outline of the design of a Decorated window, on which the centres for the tracery are formed. These centres will all be found to fall on points which, in some way or other, will be equimultiples of parts of the openings. To draw tracery well, or understand even the principles of its composition, much attention should be given to the study of the element.

Elevation.—The front façade, as the French term it, of a structure ; a geometrical drawing of the external upright parts of a building.

Embattlement.—An indented parapet ; battlement.

Emblazon.—To adorn with figures of heraldry, or ensigns armorial.

Embossing.—Sculpture in rilievo, the figures standing partly out from the plane.

Embrasure.—The opening in a battlement between the two raised solid portions or merlons, sometimes called a crenelle.

Encaustic.—Pertaining to the art of burning in colors, applied to painting on glass, porcelain, or tiles, where colors are fixed by heat ; hence, encaustic tiles, brick, etc.

Engaged Columns.—Are those attached to, or built into walls or piers, a portion being concealed.

Enrichment.—The addition of ornament, carving, etc., to plain work ; decoration ; embellishment.

Ensemble.—Means the whole work or composition considered together, and not in parts.

Entablature.—The assemblage of parts supported by the column. It consists of three parts : the architrave, frieze, and cornice.

Entail.—In Gothic architecture, delicate carving.

Entasis.—The swelling of a column, etc. In mediæval architecture, some spires, particularly those called "broach spires," have a slight swelling in the sides, but no more than to make them look straight ; for, from a particular "deceptio visus," that which is quite straight, when viewed at a height, looks hollow.

Entry.—A hall without stairs or vestibule.

Epistyle.—This term may with propriety be applied to the whole entablature, with which it is synonymous ; but it is restricted in use to the architrave, or lowest member of the entablature.

Escutcheon.—(Her.) The field or ground on which a coat-of-arms is represented. (Arch.) The shields used on tombs, in the spandrels of doors, or in

string-courses; also, the ornamented plates from the centre of which door rings, knockers, etc., are suspended, or which protect the wood of the key-hole from the wear of the key. In mediæval times these were often worked in a very beautiful manner.

Etching.—A mode of engraving on glass or metal (generally copper) by means of lines, eaten in or corroded by means of some strong acid.

Eustyle.—A species of intercolumniation to which a proportion of two diameters and a quarter is assigned. This term, together with the others of similar import—pycnostyle, systyle, diastyle, and aræostyle—referring to the distances of columns from one another in composition, is from Vitruvius, who assigns to each the space it is to express. It will be seen, however, by reference to them individually, that the words themselves, though perhaps sufficiently applicable, convey no idea of an exactly defined space, and, by reference to the columnar structures of the ancients, that no attention was paid by them to such limitations. It follows, then, that the proportions assigned to each are purely conventional, and may or may not be attended to without vitiating the power of applying the terms. Eustyle means the best or most beautiful arrangement; but, as the effect of a columnar composition depends on many things besides the diameter of the columns, the same proportioned intercolumniation would look well or ill according to those other circumstances, so that the limitation of Eustyle to two diameters and a quarter is absurd.

Extrados.—The exterior or convex curve forming the upper line of the arch stones; the term is opposed to the intrados, or concave side.

Eye of a Dome.—The aperture at its summit.

Eye of a Volute.—The circle in its centre.

Facade, or **Face.**—The whole exterior side of a building that can be seen at one view; strictly speaking, the principal front.

Face Mould.—The pattern for marking the plank or board out of which ornamental hand-railings for stairs and other works are cut.

Fan Tracery.—The very complicated mode of roofing used in the Perpendicular style, in which the vault is covered by ribs and veins of tracery.

Fascia.—A flat, broad member in the entablature of columns or other parts of buildings, but of small projection. The architraves in some of the orders are composed of three bands, or fasciæ: the Tuscan and the Doric ought to have only one. Ornamental projections from the walls of brick buildings over any of the windows, except the uppermost, are called Fasciæ.

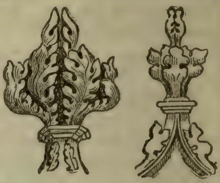
Fenestral.—A frame, or "chassis," on which oiled paper or thin cloth was strained to keep out wind and rain when the windows were not glazed.

Festoon.—An ornament of carved work, representing a wreath or garland of flowers or leaves, or both, interwoven with each other. It is thickest in the middle, and small at each extremity, where it is tied, a part often hanging down below the knot.

FESTOON.

Fillet.—A narrow vertical band or listel of frequent use in congeries of mouldings, to separate and combine them, and also to give breadth and firmness to the upper edge of a crowning cyma or cavetto, as in an external cornice. The narrow slips or breadth between the flutes of Corinthian and Ionic columns are also called fillets. In mediæval work the fillet is a small, flat, projecting square, chiefly used to separate hollows and rounds, and often found in the outer parts of shafts and boutels. In this situation the centre fillet has been termed a keel, and the two side ones, wings; but, apparently, this is not an ancient usage.

Finial.—The flower, or bunch of flowers, with which a spire, pinnacle, gablet, canopy, etc., generally terminates. Where there are crockets, the finial generally bears as close a resemblance as possible to them in point of design. They are found in early work where there are no crockets. The simplest form more resembles a bud about to burst than an open flower. They soon became more elaborate, as at Lincoln, and still more, as at Westminster and the Hôtel Cluny at Paris. Many perpendicular finials are like four crockets bound together. Almost every known example of a finial has a sort of necking separating it from the parts below.



FINIALS.

Fish-joint.—A splice where the pieces are joined butt end to end, and are connected by pieces of wood or iron placed on each side and firmly bolted to the timbers, or pieces joined. (See Chapter XXIX.)

Flags.—Flat stones, from 1 to 3 inches thick, for floors.

Flamboyant.—A name applied to the Third Pointed style in France, which seems to have been developed from the Second, as the English Perpendicular was from the Decorated. The great characteristic is, that the element of the tracery flows upward in long wavy divisions like flames of fire. In most cases, also, every division has only one cusp on each side, however long the division may be. The mouldings seem to be as much inferior to those of the preceding period as the Perpendicular mouldings were to the Early English, a fact which seems to show that the decadence of Gothic architecture was not confined to one country.

Flange.—A projecting edge, rib, or rim. Flanges are often cast on the top or bottom of iron columns, to fasten them to those above or below; the top and bottom of I-beams and channels are called the flange.

Flashings.—Pieces of lead, tin, or copper, let into the joints of a wall so as to lap over gutters or other pieces; also, pieces worked in the slates or shingles around dormers, chimneys, and any rising part, to prevent leaking.

Flatting.—Painting finished without leaving a gloss on the surface.

Flèche.—A general term in French architecture for a spire, but more particularly used for the small, slender erection rising from the intersection of the nave and transepts in cathedrals and large churches, and carrying the sanctus bell.

Flight.—A run of steps or stairs from one landing to another.

Floating.—The equal spreading of plaster or stucco on the surface of walls, by means of a board called a float; as a rule, only rough plastering is floated.

Floriated.—Having florid ornaments, as in Gothic pillars.

Fleur-de-lis.—The royal insignia of France, much used in decoration.

Flue.—The space or passage in a chimney through which the smoke ascends. Each passage is called a flue, while all together make the chimney.

Flush.—The continued surface, in the same plane, of two contiguous masses.

Flute.—A concave channel. Columns whose shafts are channelled are said to be fluted, and the flutes are collectively called Flutings.

Flying Buttress.—An arched buttress used when extra strength was required for the upper part of the wall of the nave, etc., to resist the outward thrust of a vaulted ceiling. The flying buttress generally rests on the wall and buttress of the aisle.

Foils.—The small arcs in the tracery of Gothic windows, panels, etc.

Foliage.—An ornamental distribution of leaves on various parts of buildings.

Foliation.—The use of small arcs or foils in forming tracery.

Font.—The vessel used in the rite of baptism. The earliest extant is supposed

to be that in which Constantine is said to have been baptized ; this is a porphyry labrum from a Roman bath. Those in the baptisteries in Italy are all large, and were intended for immersion ; as time went on, they seem to have become smaller. Fonts are sometimes mere plain hollow cylinders, generally a little smaller below than above ; others are massive squares, supported on a thick stem, round which sometimes there are smaller shafts. In the Early English this form is still pursued, and the shafts are detached ; sometimes, however, they are hexagonal and octagonal, and in this and the later styles assume the form of a vessel on a stem. Norman fonts have frequently curious carvings on them, approaching the grotesque ; in later times the foliages, etc., partook absolutely of the character of those used in other architectural details of their respective periods. The font in European churches is usually placed close to a pillar near the entrance, generally that nearest but one to the tower in the south arcade ; or, in large buildings, in the middle of the nave, opposite the entrance porch, and sometimes in a separate building. In Protestant churches in this country, the font is generally placed inside the communion rail, or on the steps of the chancel.

Footings.—The spreading courses at the base or foundation of a wall. When a layer of different material from that of the wall (as a bed of concrete) is used, it is called the Footing.

Foundation.—That part of a building or wall which is below the surface of the ground.

Foxtail Wedging.—Is a peculiar mode of mortising, in which the end of the tenon is notched beyond the mortise, and is split and a wedge inserted, which, being forcibly driven in, enlarges the tenon and renders the joint firm and immovable.

Frame.—The name given to the wood-work of windows, doors, etc. ; and in carpentry, to the timber works supporting floors, roofs, etc.

Framing.—The rough timber work of a house, including the flooring, roofing, partitioning, ceiling, and beams thereof.

Freestone.—Stone which can be used for mouldings, tracery, and other work required to be executed with the chisel. The oölitic and sandstones are those generally included by this term.

Fresco.—The method of painting on a wall while the plastering is wet. The color penetrates through the material, which, therefore, will bear rubbing or cleaning to almost any extent. The transparency, the chiaro-oscuro, and lucidity, as well as force, which can be obtained by this method, cannot be conceived unless the frescos of Fra Angelico or Raffaelle are studied. The word, however, is often applied improperly to painting on the surface in distemper or body color, mixed with size or white of egg, which gives an opaque effect.

Fret.—An ornament consisting of small fillets intersecting each other at right angles.

FRET.

Frieze.—That portion of an entablature between the cornice above and architrave below. It derives its name from being the recipient of the sculptured enrichments either of foliage or figures which may be relevant to the object of the sculpture. The frieze is also called the Zoöphorus.

Frigidarium.—An apartment in the Roman bath, supplied with cold water.

Furniture.—A name given to the metal trimmings of doors, windows, and other similar parts of a house. In this country the word "hardware" is more generally used to denote the same thing.

Furrings.—Flat pieces of timber used to bring an irregular framing to an even surface.

Gable.—When a roof is not hipped or returned on itself at the ends, its ends are stopped by carrying up the walls under them in the triangular form of the roof itself. This is called the gable, or, in the case of the ornamental and ornamented gable, the pediment. Of necessity, gables follow the angles of the slope of the roof, and differ in the various styles. In Norman work they are generally about half-pitch ; in Early English, seldom less than equilateral, and often more. In Decorated work they become lower, and still more so in the Perpendicular style. In all important buildings they are finished with copings or parapets. In the Later Gothic styles gables are often surmounted with battlements, or enriched with crockets ; they are also often panelled or perforated, sometimes very richly. The gables in ecclesiastical buildings are mostly terminated with a cross ; in others, by a finial or pinnacle. In later times the parapets or copings were broken into a sort of steps, called corbie steps. In buildings of less pretension the tiles or other roof covering passed over the front of the wall, which then, of course, had no coping. In this case, the outer pair of rafters were concealed by moulded or carved verge boards.

Gable Window.—A term sometimes applied to the large window under a gable, but more properly to the windows in the gable itself.

Gabled Towers.—Those which are finished with gables instead of parapets. Many of the German Romanesque towers are gabled.

Gablets.—Triangular terminations to buttresses, much in use in the Early English and Decorated periods, after which the buttresses generally terminate in pinnacles. The Early English gablets are generally plain, and very sharp in pitch. In the Decorated period they are often enriched with panelling and crockets. They are sometimes finished with small crosses, but oftener with finials.

Gain.—A bevelled shoulder on the end of a mortised brace, for the purpose of giving additional resistance to the shoulder.

Gallery.—Any long passage looking down into another part of a building, or into the court outside. In like manner, any stage erected to carry a rood or an organ, or to receive spectators, was latterly called a gallery, though originally a loft. In later times the name was given to any very long rooms, particularly those intended for purposes of state, or for the exhibition of pictures.

Gambrel Roof.—A roof with two pitches, similar to a mansard or curb roof.

Gargoyle, or Gurgoyle.—The carved termination to a spout which conveyed away the water from the gutters, supposed to be called so from the gurgling noise made by the water passing through it. Gargoyles are mostly grotesque figures.

GARGOYLE.

Gate-house.—A building forming the entrance to a town, the door of an abbey, or the enceinte of a castle or other important edifice. They generally had a large gateway protected by a gate, and also a portcullis, over which were battlemented parapets with holes (machicolations) for throwing down darts, melted lead, or hot sand on the besiegers. Gate-houses always had a lodge, with apartments for the porter, and guard-rooms for the soldiers ; and, generally, rooms over for the officers, and often places for prisoners beneath. The name is now commonly applied to the gate-keeper's lodge on large estates.

Gauge.—1. To mix plaster of Paris with common plaster to make it set quick, called gauged mortar. 2. A tool used by carpenters, to strike a line parallel to the edge of a board.

Girder.—A large timber or iron beam, either single or built up, used to support joists or walls over an opening.

Glyph.—A vertical channel in a frieze.

Gothic Style.—The name of Gothic was given to the various Mediæval styles at a period in the sixteenth century when a great classic revival was going on, and everything not classic was considered barbarian, or Gothic. The term was thus originally intended as one of stigma, and, although it conveys a false idea of the character of the Mediæval styles, it has long been used to distinguish them from the Grecian and Roman. The true principle of Gothic architecture is the vertical division, relation and subordination of the different parts, distinct and yet at unity with each other, and while this principle was adhered to, Gothic architecture may be said to have retained its vitality.

Grange.—A word derived from the French, signifying a large barn or granary. Granges were usually long buildings with high wooden roofs, sometimes divided by posts or columns into a sort of nave and aisles, with walls strongly buttressed. In England the term was applied not only to the barns, but to the whole of the buildings which formed the detached farms belonging to the monasteries; in most cases there was a chapel either included among these or standing apart as a separate edifice.

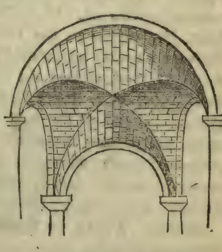
Grillage.—A framework of beams laid longitudinally and crossed by similar beams notched upon them, used to sustain walls to prevent irregular setting.

Grille.—The iron-work forming the enclosure screen to a chapel, or the protecting railing to a tomb or shrine; more commonly found in France than in England. They are of wrought iron, ornamented by the swage and punch, and put together either by rivets or clips. In modern times grilles are used extensively for protecting the lower windows in city houses, also the glass opening in outside doors.

Groin.—By some described as the line of intersection of two vaults where they cross each other, which others call the groin point; by others the curved section or spandrel of such vaulting is called a groin, and by others the whole system of vaulting is so named.

Groin Arch.—The cross-rib in the later styles of groining, passing at right angles from wall to wall, and dividing the vault into bays or travees.

Groin Ceiling.—A ceiling to a building composed of oak ribs, the spandrels of which are filled in with narrow, thin slips of wood. There are several in England; one at the Early English church at Warmington, and one at Winchester Cathedral, exactly resembling those of stone.



GROINED VAULTING.

Groin Centring.—In groining without ribs, the whole surface is supported by centring during the erection of the vaulting. In ribbed work the stone ribs only are supported by timber ribs during the progress of the work, any light stuff being used while filling in the spandrels.

Groin Point.—The name given by workmen to the arris or line of intersection of one vault with another where there are no ribs.

Groin Rib.—The rib which conceals the groin point or joints, where the spandrels intersect.

Groined Vaulting.—The system of covering a building with stone vaults which cross and intersect each other, as opposed to the barrel vaulting, or series of arches placed side by side. The earliest groins are plain, without any ribs,

except occasionally a sort of wide band from wall to wall, to strengthen the construction. In later Norman times ribs were added on the line of intersection of the spandrels, crossing each other, and having a boss as a key common to both; these ribs the French anthors call *nerfs en ogive.* Their introduction, however, caused an entire change in the system of vaulting; instead of arches of uniform thickness and great weight, these ribs were first put up as the main construction, and spandrels of the lightest and thinnest possible material placed upon them, the haunches only being loaded sufficiently to counterbalance the pressure from the crown. Shortly after, half-ribs against the walls (formerets) were introduced to carry the spandrels without cutting into the walling, and to add to the appearance. The work was now not treated as continued vaulting, but as divided into bays, and it was formed by keeping up the ogive, or intersecting ribs and their bosses; a sort of construction having some affinity to the dome was formed, which added much to the strength of the groining. Of course, the top of the soffit or ridge of the vault was not horizontal, but rose from the level of the top of the formeret-rib to the boss and fell again; but this could not be perceived from below. As this system of construction got more into use, and as the vaults were required to be of greater span and of higher pitch, the spandrels became larger, and required more support. To give this, another set of ribs was introduced, passing from the springers of the ogive ribs, and going to about half-way between these and the ogive, and meeting on the ridge of the vault; these intermediate ribs are called by the French *tiercerons*, and began to come into use in the transition from Early English to Decorated. About the same period a system of vaulting came into use called *hexpartite*, from the fact that every bay is divided into six compartments instead of four. It was invented to cover the naves of churches of unusual width. The filling of the spandrels in this style is very peculiar, and, where the different compartments meet at the ridge, some pieces of harder stone have been used, which give rather a pleasing effect. The arches against the wall, being of smaller span than the main arches, cause the centre springers to be perpendicular and parallel for some height, and the spandrels themselves are very hollow. As styles progressed, and the desire for greater richness increased, another series of ribs, called *liernes*, was introduced; these passed crossways from the *ogives* to the *tiercerons*, and thence to the *doubleaux*, dividing the spandrels nearly horizontally. These various systems increased in the Perpendicular period, so that the vaults were quite a net-work of ribs, and led at last to the Tudor, or, as it is called by many, fan-tracery vaulting. In this system the ribs are no part of the real construction, but are merely carved upon the voussoirs, which form the actual vaulting. Fan Tracery is so called because the ribs radiate from the springers, and spread out like the sticks of a fan. These later methods are not strictly groins, for the pendentives are not square on plan, but circular, and there is, therefore, no arris intersection or groin point.

Groins, Welsh, or Underpitch.—When the main longitudinal vault of any groining is higher than the cross or transverse vaults which run from the windows, the system of vaulting is called underpitch groining, or, as termed by the workmen, Welsh groining. A very fine example is at St. George's Chapel, Windsor, England.

Groove.—In joinery, a term used to signify a sunk channel whose section is rectangular. It is usually employed on the edge of a moulding, stile, or rail, etc., into which a tongue corresponding to its section, and in the substance of the wood to which it is joined, is inserted.

Grotesque.—A singular and fantastic style of ornament found in ancient buildings.

Grotto.—An artificial cavern.

Ground Floor.—The floor of a building on a level, or nearly so, with the ground.

Ground Joist.—Joist that is blocked up from the ground.

Grounds.—Pieces of wood embedded in the plastering of walls to which skirting and other joiner's work is attached. They are also used to stop the plastering around door and window openings.

Grouped Columns.—Three, four, or more columns put together on the same pedestal. When two are placed together, they are said to be coupled.

Grout.—Mortar made so thin by the addition of water that it will run into all the joints and cavities of the mason-work, and fill it up solid.

GUILLOCHE.

Guilloche, or Guillochos.—An interlaced ornament like net-work, used most frequently to enrich the torus.

Guttæ.—The small cylindrical drops used to enrich the mutules and regulæ of the Doric entablature are so called.

GUTTÆ.

Gutter.—The channel for carrying off rain-water. The mediæval gutters differed little from others, except that they are often hollows sunk in the top of stone cornices, in which case they are generally called channels in English, and *cheneaux* in French.

Gymnasium.—A building classed in the first rank by the Greeks; it was in them they instructed the youth in all the arts of peace and war; a building for athletic exercises.

Hall.—1. The principal apartment in the large dwellings of the Middle Ages, used for the purposes of receptions, feasts, etc. In the Norman castle the hall was generally in the keep above the ground floor, where the retainers lived, the basement being devoted to stores and dungeons for confining prisoners. Later halls—indeed, some Norman halls (not in castles)—are generally on the ground floor, as at Westminster, approached by a porch either at the end, as in this last example, or at the side, as at Guildhall, London, having at one end a raised dais or estrade. The roofs are generally open and more or less ornamented. In the middle of these was an opening to let out the smoke, though in later times the halls have large chimney-places with funnels or chimney-shafts for this purpose. At this period there were usually two deeply recessed bay windows at each end of the dais, and doors leading into the withdrawing-rooms, or the ladies' apartments; they are also generally wainscoted with oak, in small panels, to the height of five or six feet, the panels often being enriched. Westminster Hall was originally divided into three parts, like a nave and side aisles, as are some on the Continent of Europe. 2. A room or passage-way at the entrance of a house, or suite of chambers. 3. A place of public assembly, as a town-hall, a music-hall.

Halving.—The junction of two pieces of timber, by letting one into the other.

Hammer Beam.—A beam in a Gothic roof, not extending to the opposite side; a beam at the foot of a rafter.

Hanging Buttress.—A buttress not rising from the ground, but supported on a corbel, applied chiefly as a decoration and used only in the Decorated and Perpendicular style.

Hanging Stile.—Of a door, is that to which the hinges are fixed.

Hangings.—Tapestry; originally invented to hide the coarseness of the

walls of a chamber. Different materials were employed for this purpose, some of them exceedingly costly and beautifully worked in figures, gold and silk.

Hatching.—Drawing parallel lines close together for the purpose of indicating a section of anything. The lines are generally drawn at an angle of 45° with a horizontal.

Haunches.—The sides of an arch, about half-way from the springing to the crown.

Headers.—In masonry, are stones or bricks extending over the thickness of a wall. In carpentry, the large beam into which the common joists are framed in framing openings for stairs, chimneys, etc.

Heading Courses.—Courses of a wall in which the stone or brick are all headers.

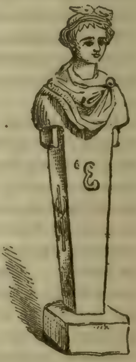
Head-way.—Clear space or height under an arch, or over a stairway, and the like.

Heel.—Of a rafter, the end or foot that rests upon the wall plate.

Height.—Of an arch, a line drawn from the middle of the chord to the intrados.

Helix.—A small volute or twist like a stalk, representing the twisted tops of the acanthus, placed under the abacus of the Corinthian capital.

Hermes.—A rough quadrangular stone or pillar, having a head, usually of Hermes or Mercury, sculptured on the top, without arms or body, placed by the Greeks in front of buildings.


HERMES.

Herring-bone Work.—Bricks, tile, or other materials arranged diagonally in building.

Hexastyle.—A portico of six columns in front is of this description.

High Altar.—The principal altar in a cathedral or church. Where there is a second, it is generally at the end of the choir or chancel, not in the lady chapel.

Hip-knob.—The finial on the hip of a roof, or between the barge boards of a gable.

Hip-roof.—A roof which rises by equally inclined planes from all four sides of the building.

Hippodrome.—A place appropriated by the ancients for equestrian exercises.

Hips.—Those pieces of timber placed in an inclined position at the corners or angles of a hip-roof.

Hood-mould.—A word used to signify the drip-stone for label over a window or door opening, whether inside or out.

Hôtel de Ville.—The town-hall, or guild-hall, in France, Germany, and Northern Italy. The building, in general, serves for the administration of justice, the receipt of town dues, the regulation of markets, the residence of magistrates, barracks for police, prisons, and all other fiscal purposes. As may be imagined, they differ very much in different towns, but they have almost invariably attached to them, or closely adjacent, a large clock-tower containing one or more bells, for calling the people together on special occasions.

Hôtel Dieu.—The name for a hospital in mediæval times. In England there are but few remains of these buildings, one of which is at Dover ; in France there are many. The most celebrated is the one at Angers, described by Parker. They do not seem to differ much in arrangement of plan from those in modern days, the accommodation for the chaplain, medicine, nurses, stores, etc., being much the same in all ages, except that in some of the earlier, instead of the sick

being placed in long wards like galleries, as is now done, they occupied large buildings, with naves and side aisles, like churches.

Housing.—The space taken out of one solid to admit the insertion of another. The base on a stair is generally housed into the treads and risers ; a niche for a statue.

Hypæthros.—A temple open to the air, or uncovered. The term may be the more easily understood by supposing the roof removed from over the nave of a church in which columns or piers go up from the floor to the ceiling, leaving the aisles still covered.

Hypogea.—Constructions under the surface of the earth, or in the sides of a hill or mountain.

Ichnography.—A horizontal section of a building or other object, showing its true dimensions according to a geometric scale ; a ground plan.

Impluvium.—The central part of an ancient Roman court, which was uncovered.

Impost.—A term in classic architecture for the horizontal mouldings of piers or pilasters, from the top of which spring the archivolts or mouldings which go round the arch.

In Antis.—When there are two columns between the antæ of the lateral walls and the cella.

Incise.—To cut in ; to carve : to engrave.

Indented.—Toothed together.

Inlaying.—Inserting pieces of ivory, metal, or choice woods, or the like, into a groundwork of some other material, for ornamentation.

Insulated.—Detached from another building. A church is insulated, when not contiguous to any other edifice. A column is said to be insulated, when standing free from the wall ; thus, the columns of peripteral temples were insulated.

Intaglio.—A sculpture or carving in which the figures are sunk below the general surface, such as a seal the impression of which in wax is in bas-relief ; opposed to Cameo.

Intercolumniation.—The distance from column to column, the clear space between columns.

Interlaced Arches.—Arches where one passes over two openings, and they consequently cut or intersect each other.

Intrados.—Of an arch, the inner or concave curve of the arch stones.

Inverted Arches.—Those whose key-stone or brick is the lowest in the arch.

Ionic Order.—One of the orders of Classical architecture.

Iron Work.—In mediæval architecture, as an ornament, is chiefly confined to the hinges, etc., of doors and of church chests, etc. In some instances not only do the hinges become a mass of scroll work, but the surface of the doors is covered by similar ornaments. In almost all styles the smaller and less important doors had merely plain strap hinges, terminating in a few bent scrolls, and latterly in fleurs-de-lis. Escutcheon and ring handles, and the other furniture, partook more or less of the character of the time. On the Continent of Europe the knockers are very elaborate. At all periods doors have been ornamented with nails having projecting heads, sometimes square, sometimes polygonal, and sometimes ornamented with roses, etc. The iron work of windows is generally plain, and the ornament confined to simple fleur-de-lis heads to the stanchions. The iron work of screens enclosing tombs and chapels is noticed under *Grille*, *q. v.*

Jack.—An instrument for raising heavy loads, either by a crank, siren and pinion, or by hydraulic power, and in all cases worked by hand.

Jack Rafter.—A short rafter, used especially in hip-roofs.

Jamb.—The side-post or lining of a doorway or other aperture. The jambs of a window outside the frame are called Reveals.

Jamb-shafts.—Small shafts to doors and windows with caps and bases; when in the inside arris of the jamb of a window they are sometimes called Escon-sons.

Joggle.—A joint between two bodies so constructed by means of jogs or notches as to prevent their sliding past each other.

Joinery.—That branch in building confined to the nicer and more ornamental parts of carpentry.

Joist.—A small timber to which the boards of a floor or the laths of ceiling are nailed. It rests on the wall or on girders.

Keep.—The inmost and strongest part of a mediæval castle, answering to the citadel of modern times. The arrangement is said to have originated with Gundolf, the celebrated Bishop of Rochester. The Norman keep is generally a very massive square tower, the basement or stories partly below ground being used for stores and prisons. The main story is generally a great deal above ground level, with a projecting entrance, approached by a flight of steps and drawbridge. This floor is generally supposed to have been the guard-room or place for the soldiery; above this was the hall, which generally extended over the whole area of the building, and is sometimes separated by columns; above this are other apartments for the residents. There are winding staircases in the angles of the buildings, and passages and small chambers in the thickness of the walls. The keep was intended for the last refuge, in case the outworks were scaled and the other buildings stormed. There is generally a well in a mediæval keep, ingeniously concealed in the thickness of a wall, or in a pillar. The most celebrated of Norman times are the White Tower in London, the castles at Rochester, Arundel, and Newcastle, Castle Hedingham, etc. The keep was often circular.

Key-stone.—The stone placed in the centre of the top of an arch. The character of the key-stone varies in different orders. In the Tuscan and Doric it is only a simple stone projecting beyond the rest; in the Ionic it is adorned with mouldings in the manner of a console; in the Corinthian and Composite it is a rich-sculptured console.

King-post.—The middle post of a trussed piece of framing for supporting the tie-beam at the middle and the lower ends of the struts.

Knee.—A piece of timber naturally or artificially bent to receive another to relieve a weight or strain.

Knob, Knot.—The bunch of flowers carved on a corbel, or on a Boss.

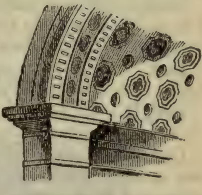
Kremlin.—The Russian name for the citadel of a town or city.

Label.—Gothic: the drip or hood-moulding of an arch, when it is returned to the square.

Label Terminations.—Carvings on which the labels terminate near the springing of the windows. In Norman times those were frequently grotesque heads of fish, birds, etc., and sometimes stiff foliage. In the Early English and Decorated periods they are often elegant knots of flowers, or heads of kings, queens, bishops, and other persons supposed to be the founders of churches. In the Perpendicular period they are often finished with a short square, mitred return or knee, and the foliages are generally leaves of square or octagonal form.

Lacunar.—A panelled or coffered ceiling or soffit. The panels or cassoons of a ceiling are by Vitruvius called lacunaria.

Lady-chapel.—A small chapel dedicated to the Virgin Mary, generally found in ancient cathedrals.



LACUNARS IN CEILING.

Lancet.—A high and narrow window pointed like a lancet, often called a lancet window.

Landing.—A platform in a flight of stairs between two stories; the terminating of a stair.

Lantern.—A turret raised above a roof or tower and very much pierced, the better to transmit light. In modern practice this term is generally applied to any raised part in a roof or ceiling containing vertical windows, but covered in horizontally. The name was also often applied to the louver or femerell on a roof to carry off the smoke; sometimes, too, to the open constructions at the top of towers, as at Ely Cathedral, probably because lights were placed in them at night to serve as beacons.

Lanterns of the Dead.—Curious small slender towers, found chiefly in the centre and west of France, having apertures at the top, where a light was exhibited at night to mark the place of a cemetery. Some have supposed that the round towers in Ireland may have served for this purpose.

Lath.—A slip of wood used in slating, tiling, and plastering.

Lattice.—Any work of wood or metal made by crossing laths, rods, or bars, and forming a net-work. 2. A reticulated window, made of laths or slips of iron, separated by glass windows, and only used where air rather than light is to be admitted, as in cellars and dairies.

Lavabo.—The lavatory for washing hands, generally erected in cloisters of monasteries. A very curious one at Fontenay, surrounding a pillar, is given by Viollet le-Duc. In general, it is a sort of trough, and in some places has an almry for towels, etc.

Lavatory.—A place for washing the person.

Lean-to.—A small building whose rafters pitch or lean against another building, or against a wall.

Lectern.—The reading-desk in the choir of churches.

Ledge, or Ledgement.—A projection from a plane, as slips on the side of window and door frames to keep them steady in their places.

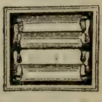
Ledgers.—The horizontal pieces fastened to the standard poles or timbers of scaffolding raised around buildings during their erection. Those which rest on the ledgers are called putlogs, and on these the boards are laid.

Lewis.—An iron clamp dovetailed into a large stone to lift it by.

Lich-gate.—A covered gate at the entrance of a cemetery, under the shelter of which the mourners rested with the corpse, while the procession of the clergy came to meet them. There are several examples in England.

Light.—A division or space in a sash for a single pane of glass; also a pane of glass.

Linen Scroll.—An ornament formerly used for filling panels, and so called from its resemblance to the convolutions of a folded napkin.



LINEN SCROLL.

Lining.—Covering for the interior, as casing is covering the exterior surface of a building; also, such as linings of a door for windows, shutters, and similar work.

Lintel.—The horizontal piece which covers the opening of a door or window.

Lip Mould.—A moulding of the Perpendicular period like a hanging lip.

List, or Listel.—A little square moulding, to crown a larger; also termed a fillet.

Lithograph.—A print from a drawing on stone.

Lobby.—An open space surrounding a range of chambers, or seats in a theatre; a small hall or waiting room.

Lodge.—A small house in a park.

Loft.—The highest room in a house, particularly if in the roof ; also, a gallery raised up in a church to contain the rood, the organ, or singers.

Loggia.—An outside gallery or portico above the ground, and contained within the building.

Loop-hole.—An opening in the wall of a building, very narrow on the outside, and splayed within, from which arrows or darts might be discharged on an enemy. They are often in the form of a cross, and generally have round holes at the ends.

Lombard Architecture.—A name given to the round-arched architecture of Italy, introduced by the conquering Goths and Ostrogoths, and which superseded the Romanesque. It reigned between the eighth and twelfth centuries, during the time that the Saxon and Norman styles were in vogue in England, and corresponded with them in its development into the Continental Gothic.

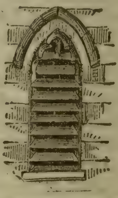
Lotus.—A plant of great celebrity amongst the ancients, the leaves and blossoms of which generally form the capitals of Egyptian columns.

Louver.—A kind of vertical window, frequently in the peaks of gables, and in the top of towers, and provided with horizontal slats which permit ventilation and exclude rain.

Lozenge Moulding.—A kind of moulding used in Norman architecture, of many different forms, all of which are characterized by lozenge-shaped ornaments.

Lunette.—The French term for the circular opening in the groining of the lower stories of towers, through which the bells are drawn up.

LOZENGE MOULDING.



LOUVER WINDOW.

Machicolation.—A parapet or gallery projecting from the upper part of the wall of a house or fortification, supported by brackets or corbels, and perforated in the lower part so that the defenders of the building might throw down darts, stones, and sometimes hot sand, molten lead, etc., upon their assailants below.

Man-hole.—A hole through which a man may creep into a drain, cesspool, steam-boiler, etc.

Manor-house.—The residence of the suzerain or lord of the manor ; in France the central tower or keep of a castle is often called the *manoir*.

Mansard Roof.—Curb roof, invented by François Mansard, a distinguished French architect, who died in 1666.

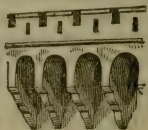
Mansion.—A residence of considerable size and pretension.

Mantel.—The work over a fireplace in front of a chimney ; especially, a shelf, usually ornamented, above the fireplace.

Marquetry.—Inlaid work of fine hard pieces of wood of different colors; also of shells, ivory, and the like.

Mausoleum.—A magnificent tomb or sumptuous sepulchral monument.

Medallion.—Any circular tablet on which are embossed figures or busts.



MACHICOLATION.

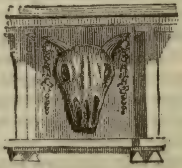
Mediæval Architecture.—The architecture of Eng-

land, France, Germany, etc., during the Middle Ages, including the Norman and Early Gothic styles. It comprises also the Romanesque, Byzantine and Saracenic, Lombard, and other styles.

Members.—The different parts of a building, the different parts of an entablature, the different mouldings of a cornice, etc.

Merlon.—That part of a parapet which lies between two embrasures.

Metope.—The square recess between the triglyphs in a Doric frieze. It is sometimes occupied by sculptures.



METOPE.

Mezzanine.—A low story between two lofty ones. It is called by the French *entresol*, or inter-story.

Mezzo-rilievo.—Or mean relief, in comparison with alto-rilievo, or high relief.

Minaret.—Turkish: a circular turret rising by different stages or divisions, each of which has a balcony.

Minster.—Probably a corruption of monasterium—the large church attached to any ecclesiastical fraternity. If the latter be presided over by a bishop, it is generally called a Cathedral; if by an abbot, an Abbey; if by a prior, a Priory.

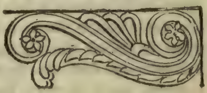
Minute.—The sixtieth part of the lower diameter of a column; it is the measure used by architects to determine the proportions of an order.

MINARET.

Miserere.—A seat in a stall of a large church made to turn up and afford support to a person in a position between sitting and standing. The under side is generally carved with some ornament, and very often with grotesque figures and caricatures of different persons.

Mitre.—A moulding returned upon itself at right angles is said to mitre. In joinery, the ends of any two pieces of wood of corresponding form, cut off at 45°, necessarily abut upon one another so as to form a right angle, and are said to mitre.

Modillion.—So called because of its arrangement in regulated distances; the enriched block or horizontal bracket generally found under the cornice of the Corinthian entablature. Less ornamented, it is sometimes used in the Ionic.



MODILLION.

Module.—This is a term which has been generally used by architects in determining the relative proportions of the various parts of a columnar ordinance. The semi-diameter of the column at its base is the module, which being divided into thirty parts called minutes, any part of the composition is said to be of so many modules and minutes, or minutes alone, in height, breadth, or projection. The whole diameter is now generally preferred as a module, it being a better rule of proportion than its half.

Monastery.—A set of buildings adapted for the reception of any of the various orders of monks, the different parts of which are described in the separate article, *Abbey*.

Monotriglyph.—The intercolumniations of the Doric order are determined by the number of triglyphs which intervene, instead of the number of diameters of the column, as in other cases; and this term designates the ordinary intercolumniation of one triglyph.

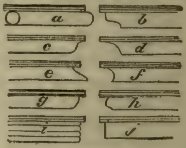
Monument.—A name given to a tomb, particularly to those fine structures recessed in the walls of mediæval churches.

Mosaic.—Pictorial representations, or ornaments, formed of small pieces of stone, marble, or enamel of various colors. In Roman houses the floors are often

entirely of mosaic, the pieces being cubical. The best examples of mosaic work are found in St. Mark's, at Venice.

Mosque.—A Mahometan temple, or place of worship.

Moulding.—When any work is wrought into long regular channels or projections, forming curves or rounds, hollows, etc., it is said to be moulded, and each separate member is called a moulding. In mediæval architecture the principal mouldings are those of the arches, doors, windows, piers, etc. In the Early English style, the mouldings, for some time, formed groups set back in squares, and frequently very deeply undercut. The scroll moulding is also common. Small fillets now become very frequent in the keel moulding, from its resemblance in section to the bottom of a ship; sometimes, also, it has a peculiar hollow on each side, like two wings. Later in the Decorated style the mouldings are more varied in design, though hollows and rounds still prevail. The undercutting is not so deep, fillets abound, ogees are more frequent, and the wave mould, double ogee, or double ressaunt, is often seen. In many places the strings and labels are a round, the lower half of which is cut off by a plain chamfer. The mouldings in the later styles in some degree resemble those of the Decorated, flattened and extended; they run more into one another, having fewer fillets, and being, as it were, less grouped. One of the principal features of the change is the substitution of one, or perhaps two (seldom more), very large hollows in the set of mouldings. These hollows are neither circular nor elliptical, but obovate, like an egg cut across, so that one half is larger than the other. The brace mould also has a small bead, where the two ogees meet. Another sort of moulding, which has been called a lip mould, is common in parapets, bases, and weatherings.



MOULDINGS.

a, astragal; *b*, ogee; *c*, cymatium; *d*, cavetto; *e*, scotia, or casement; *f*, apophyges; *g*, ovolo, or quarter round; *h*, torus; *i*, reeding; *j*, band.

Mouldings, Ornamented.—The Saxon and early Norman mouldings do not seem to have been much enriched, but the complete and later styles of Norman are remarkable for a profusion of ornamentation, the most usual of which is what is called the zigzag. This seems to be to Norman architecture what the meander or fret was to the Grecian; but it was probably derived from the Saxons, as it is very frequently found in their pottery. Bezants, quatrefoils, lozenges, crescents, billets, heads of nails, are very common ornaments. Besides these, battlements, cables; large ropes round which smaller ropes are turned, or, as our sailors say, "wormed"; scallops, pellets, chains, a sort of conical barrels, quaint stiff foliages, beaks of birds, heads of fishes, ornaments of almost every conceivable kind, are sculptured in Norman mouldings; and they are used in such profusion as has been attempted in no other style. The decorations on Early English mouldings are chiefly the dog-tooth, which is one of the great characteristics of this style, though it is to be found in the Transition Norman. It is generally placed in a deep hollow between two projecting mouldings, the dark shadow in the hollow contrasting in a very beautiful way with the light in these mouldings. In this period and in the next the tympanum over doorways, particularly if they are double doors, is highly ornamented. Those of the Decorated period resemble the former, except that the foliage is more natural and the dog-tooth gives way to the ball-flower. Some of the hollows, also, are ornamented with rosettes set at intervals, which are sometimes connected by a running tendril, as the ball-flowers are frequently. Some very pleasing leaf-like ornaments in the labels of windows are often found in Continental architecture. In the Perpen-

dicular period the mouldings are ornamented very frequently by square four-leaved flowers set at intervals, but the two characteristic ornaments of the time are running patterns of vine leaves, tendrils, and grapes in the hollows, which by old writers are called "vignettes in casements," and upright stiff leaves, generally called the Tudor leaf. On the Continent mouldings partook much of the same character.

Mullion, Munion.—The perpendicular pieces of stone, sometimes like columns, sometimes like slender piers, which divide the bays or lights of windows or screen-work from each other. In all styles, in less important work, the mullions are often simply plain chamfered, and more commonly have a very flat hollow on each side. In larger buildings there is often a bead or boutell on the edge, and often a single very small column with a capital. As tracery grew richer, the windows were divided by a larger order of mullion, between which came a lesser or subordinate set of mullions, which ran into each other. The term is also applied to a wood or iron division between two windows.

Multifoil.—A leaf ornament consisting of more than five divisions, applied to foils in windows.

Mutule.—The rectangular impending block under the corona of the Doric cornice, from which guttæ, or drops, depend. Mutule is equivalent to modillion, but the latter term is applied more particularly to enriched blocks or brackets, such as those of Ionic and Corinthian entablatures.

Narthex.—The long arcaded porch forming the entrance into the Christian basilica. Sometimes there was an inner narthex, or lobby, before entering the church. When this was the case, the former was called exo-narthex, and the latter eso-narthex. In the Byzantine churches this inner narthex forms part of the solid structure of the church, being marked off by a wall or row of columns, whereas in the Latin churches it was usually formed only by a wooden or other temporary screen.

Natural Beds.—In stratified rocks, is the surface of a stone as it lies in the quarry. If not laid in walls in their natural bed the laminæ separate.

Nave.—The central part between the arches of a church, which formerly was separated from a chancel or choir by a screen. It is so called from its fancied resemblance to a ship. In the nave were generally placed the pulpit and font. In continental Europe it often also contains a high altar, but this is of rare occurrence in England.

Necking.—The annulet or round, or series of horizontal mouldings, which separates the capital of a column from the plain part or shaft.

Newel.—In mediæval architecture, the circular ends of a winding staircase which stand over each other, and form a sort of cylindrical column.

Newel Post.—The post, plain or ornamented, placed at the first, or lowest step to receive or start the hand-rail upon.

Niche.—A recess sunk in a wall generally for the reception of a statue. Niches sometimes terminate by a simple label but more commonly by a canopy and with a bracket or corbel for the figure, in which case they are often called tabernacles.

Norman Style.—Was that species of Romanesque which was practised by the Normans, and which was introduced and fully developed in England after they had established themselves in it. The chief features of this style are plainness and massiveness. The arches, windows, and doorways were semicircular, the pillars were very massive, and often built up of small stones laid like brickwork.

Nosings.—The rounded and projecting edges of the treads of a stair, or the edge of a landing.

Obelisk.—Lofty pillars of stone, of a rectangular form, diminishing toward the top, and generally ornamented with inscriptions and hieroglyphics among the ancient Egyptians.

Observatory.—A building erected on an elevated spot of ground for making astronomical observations.

Octostyle.—A portico of eight columns in front.

Offsets.—When the face of a wall is not one continued surface, but sets in by horizontal jogs, as the wall grows higher and thinner, the jogs are called offsets.

Ogee.—The name applied to a moulding, partly a hollow and partly a round, and derived no doubt from its resemblance to an O placed over a G. It is rarely found in Norman work, and is not very common in Early English. It is of frequent use in Decorated work, where it becomes sometimes double, and is called a wave moulding; and later still, two waves are connected with a small bead, which is then called a brace moulding. In ancient MSS. it is called a Ressaunt.

Orchestra.—In ancient theatres, where the chorus used to dance; in modern theatres, where the musicians sit.

Order.—A column with its entablature and stylobate is so called. The term is the result of the dogmatic laws deduced from the writings of Vitruvius, and has been exclusively applied to those arrangements which they were thought to warrant.

Oriel Window.—Gothic: a projecting angular window, commonly of a trigonal or pentagonal form, and divided by mullions and transoms into different bays and compartments.

Orthography.—A geometrical elevation of a building or other object in which it is represented as it actually exists or may exist, and not perspectively, or as it would appear.

Orthostyle.—A columnar arrangement in which the columns are placed in a straight line.

Ovolo.—Same as *Echinus*.

Pagoda.—A name given to temples in India and China.

Palace.—The dwelling of a king, prince, or bishop.

Pale.—A fence picket, sharpened at the upper end.

Pane.—Probably a diminutive of panneau, a term applied to the different pieces of glass in a window; same as *Light*.

Panel.—Properly a piece of wood framed within four other pieces of wood, as in the styles and rails of a door, filling up the aperture, but often applied both to the whole square frame and the sinking itself; also to the ranges of sunken compartments in wainscoting, cornices, corbel tables, groined vaults, ceilings, etc.

Pantograph, or **Pentagraph.**—An instrument for copying on the same, or an enlarged or reduced scale.

Pantry.—An apartment or closet in which bread and other provisions are kept.

Papier-maché.—A hard substance made of a pulp from rags or paper mixed with size or glue, and moulded into any desired shape. Much used for architectural ornaments.

Parapet.—A dwarf wall along the edge of a roof, or round a terrace walk, etc., to prevent persons from falling over, and as a protection to the defenders in case of a siege. Parapets are either plain, embattled, perforated, or panelled. The last two are found in all styles except the Norman. Plain parapets are simply portions of the wall generally overhanging a little, with coping at the top and corbel table below. Embattled parapets are sometimes panelled, but oftener

pierced for the discharge of arrows, etc. Perforated parapets are pierced in various devices—as circles, trefoils, quatrefoils, and other designs—so that the light is seen through. Panelled parapets are those ornamented by a series of panels, either oblong or square, and more or less enriched, but are not perforated. These are common in the Decorated and Perpendicular periods.

Pargeting.—A species of plastering decorated by impressing patterns on it when wet. These seem generally to have been made by sticking a number of pins in a board in certain lines or curves, and then pressing on the wet plaster in various directions, so as to form geometrical figures. Sometimes these devices are in relief, and in the time of Elizabeth represent figures, birds, foliages, etc. 2. Rough plastering, commonly adopted for the interior surface of chimneys.

Parlor.—A room in a house which the family usually occupy for society and conversation, and for receiving visitors. 2. The apartment in a monastery or nunnery where the inmates are permitted to meet and converse with each other, or with visitors and friends from without.

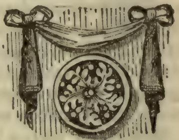
Parochial.—Belonging or relating to a parish.

Parquetry, or **Marquetry.**—A kind of inlaid floor composed of small pieces of wood either square or triangular, which are capable of forming, by their disposition, various combinations of figures; this description of joinery is very suitable for the floors of libraries, halls, and public apartments.

Party Walls.—Partitions of brick or stone between buildings on two adjoining properties.

Patera.—A circular ornament resembling a dish, often worked in relief on friezes, etc.

Pavement.—Tessellated, a pavement of mosaic work, used by the ancients, made of square pieces of stone, etc., called Tessera.



PATERA.

Pavilion.—A turret or small insulated building, and comprised beneath a single roof; also, the projecting part in front of a building which marks the centre, and which sometimes flanks a corner, when it is termed an angular pavilion.

Pedestal.—The square support of a column, statue, etc.; and the base or lower part of an order of columns: it consists of a plinth for a base, the die, and a talon crowned for a cornice. When the height and width are equal, it is termed a square pedestal · one which supports two columns, a double pedestal; and if it supports a row of columns without any break, it is a continued pedestal.

Pediment.—A low triangular crowning, ornamented, in front of a building, and over doors and windows. Pediments are sometimes made in the form of a segment; the space enclosed within the triangle is called the tympanum. Also, the gable ends of classic buildings, where the horizontal cornice is carried across the front, forming a triangle with the end of the roof.

Pendent.—A name given to an elongated boss, either moulded or foliated, such as hang down from the intersection of groins, especially in fan tracery, or at the end of hammer beams. Sometimes long corbels, under the wall pieces, have been so called. The name has also been given to the large masses depending from enriched ceilings, in the later works of the Pointed style.

Pendent Posts.—A name given to those timbers which hang down the side of a wall from the plate in hammer beam trusses, and which receive the hammer braces.

Pendentive.—A name given to an arch which cuts off, as it were, the corners of a square building internally, so that the superstructure may become an octagon

or a dome. In mediæval architecture these arches, when under a spire in the interior of a tower, are called Squinches.

Pendentive Bracketing, or **Cove Bracketing.**—Springing from the rectangular walls of an apartment upward to the ceiling, and forming the horizontal part of the ceiling into a circle or ellipse.

Pentastyle.—Having five columns in front.

Pent-roof.—A roof with a slope on one side only.

Perch.—A measure used in measuring stone work, being $24\frac{3}{4}$ cu. ft. and $16\frac{2}{3}$ cu. ft., according to locality and custom.

Periptery.—An edifice or temple surrounded by a peristyle.

Peristyle.—A range of columns encircling an edifice, such as that which surrounds the cylindrical drum under the cupola of St. Paul's. The columns of a Greek peripteral temple form a peristyle also, the former being a circular, and the latter a quadrilateral peristyle.

Perpendicular Style.—The third and last of the Pointed or Gothic styles; also called the Florid style.

Perspective Drawing.—The art of making such a representation of an object upon a plane surface as shall present precisely the same appearance that the object itself would to the eye situated at a particular point.

Pews.—A word of uncertain origin, signifying fixed seats in churches, composed of wood framing, mostly with ornamented ends. They seem to have come into general use early in the reign of Henry VI. and to have been rented and "well paid for" before the Reformation. Some bench ends are certainly of a decorated character, and some have been considered to be of the Early English period. They are sometimes of plain oak board, two and a half to three inches thick, chamfered, and with a necking and finial, generally called a poppy head; others are plainly panelled with bold cappings; in others the panels are ornamented with tracery or with the linen pattern, and sometimes with running foliages. The divisions are filled in with thin chamfered boarding, sometimes reaching to the floor, and sometimes only from the capping to the seat.

Picket.—A narrow board, often pointed, used in making fences; a pale or paling.

Pier-glass.—A mirror hanging between windows.

Piers.—The solid parts of a wall between windows, and between voids generally. The term is also applied to masses of brick-work or masonry which are insulated to form supports to gates or to carry arches, posts, girders, etc.

Pilasters.—Are flat square columns, attached to a wall, behind a column, or along the side of a building, and projecting from the wall about a fourth or a sixth part of their breadth. The Greeks had a slightly different design for the capitals of pilasters, and made them the same width at top as at bottom, but the Romans gave them the same capitals as the columns, and made them of diminished width at the top, similar to the columns.

Pile.—A large stake or trunk of a tree, driven into soft ground, as at the bottom of a river, or in made land, for the support of a building. (See p. 134.)

Pillar, or **Pyller.**—A word generally used to express the round or polygonal piers, or those surrounded with clustered columns, which carry the main arches of a building. Saxon and Early Norman pillars are generally stout cylindrical shafts built up of small stones. Sometimes, however, they are quite square, sometimes with other squares breaking out of them (this is more common in French and German work), sometimes with angular shafts, and sometimes they are plain octagons. In Romanesque Norman work the pillar is sometimes square, with two or more semicircular or half-columns attached. In the Early English period the pillars become loftier and lighter, and in most important buildings are a series

of clustered columns, frequently of marble, placed side by side, sometimes set at intervals round a circular centre, and sometimes almost touching each other. These shafts are often wholly detached from the central pillar, though grouped round it, in which case they are almost always of Purbeck or Bethersden marbles. In Decorated work the shafts on plan are very often placed round a square set anglewise, or a lozenge, the long way down the nave ; the centre or core itself is often worked into hollows or other mouldings, to show between the shafts, and to form part of the composition. In this and the latter part of the previous style there is generally a fillet on the outer part of the shaft, forming what has been called a keel moulding. They are also often, as it were, tied together by bands formed of rings of stone and sometimes of metal. The small pillars at the jambs of doors and windows, and in arcades, and also those slender columns attached to pillars, or standing detached, are generally called shafts.

Pin.—A cylindrical piece of wood, iron, or steel, used to hold two or more pieces together, by passing through a hole in each of them, as in a mortise and tenon joint, or a pin joint of a truss.

Pinnacle.—An ornament originally forming the cap or crown of a buttress or small turret, but afterward used on parapets at the corners of towers and in many other situations. It was a weight to counteract the thrust of the groining of roofs, particularly where there were flying buttresses ; it stopped the tendency to slip of the stone copings of the gables, and counterpoised the thrust of spires ; it formed the piers to steady the elegant perforated parapets of later periods ; and in France, especially, served to counterbalance the weight of overhanging corbel tables, huge gargoyles, etc. In the Early English period the smaller buttresses frequently finished with gablets, and the more important with pinnacles supported with clustered shafts. At this period the pinnacles were often supported on these shafts alone, and were open below ; and in larger work in this and the subsequent periods they frequently form niches and contain statues. In France, pinnacles, like spires, seem to have been in use earlier than in England. There are small pinnacles at the angles of the tower in the Abbey of Saintes. At Roullet there are pinnacles in a similar position, each composed of four small shafts, with caps and bases surmounted with small pyramidal spires. In all these examples the towers have semicircular headed windows.

PINNACLE.

Pitch of a Roof. — The proportion obtained by dividing the span by the height ; thus, we speak of its being one-half, one-third, one-fourth. When the length of the rafters is equal to the breadth of the building it is denominated Gothic.

Pitching-piece.—A horizontal timber, with one of its ends wedged into the wall at the top of a flight of stairs, to support the upper end of the rough strings.

Place.—An open piece of ground surrounded by buildings, generally decorated with a statue, column, or other ornament.

Plan.—A horizontal geometrical section of the walls of a building ; or indications, on a horizontal plane, of the relative positions of the walls and partitions, with the various openings, such as windows and doors, recesses and projections, chimneys and chimney-breasts, columns, pilasters, etc. This term is often incorrectly used in the sense of Design.

Planceer.—Is sometimes used in the same sense as soffit, but is more correctly applied to the soffit of the corona in a cornice.

Plastering.—A mixture of lime, hair, and sand, to cover lath-work between

timbers or rough walling, used from the earliest times, and very common in Roman work. In the Middle Ages, too, it was used not only in private, but in public constructions. On the inside face of old rubble walls it was not only used for purposes of cleanliness, rough work holding dirt and dust, but as a ground for distemper painting (tempera, or, as it is often improperly called, fresco), a species of ornament often used in the Middle Ages. At St. Albans Abbey, England, the Norman work is plastered, and covered with lines imitating the joints of stone. The same thing is found in English Perpendicular work. On the outside of rubble walls, and often of wood framing, it was used as roughcast; when ornamented in patterns outside, it is called pargeting.

Plate.—The piece of timber in a building which supports the end of the rafters.

Plinth.—The square block at the base of a column or pedestal. In a wall, the term plinth is applied to the projecting base or water table, generally at the level of the first floor.

Plumb.—Perpendicular; that is, standing according to a plumb line, as, the post of a house or wall is plumb.

Plumbing.—The lead and iron pipes and other apparatus employed in conveying water, and for toilet purposes in a building; originally the art of casting and working in lead.

Ply.—Used to denote the number of thicknesses of roofing paper, as three ply, four ply, etc.

Podium.—A continued pedestal; a projection from a wall, forming a kind of gallery.

Polytriglyph.—An intercolumniation in the Doric order of more than two triglyphs.

Poppy Heads.—Probably from the French *poupée*: the finials or other ornaments which terminate the tops of bench ends, either to pews or stalls. They are sometimes small human heads, sometimes richly carved images, knots of foliage, or finials, and sometimes fleurs-de-lis simply cut out of the thickness of the bench end and chamfered.


POPPY HEAD.

Porch.—A covered erection forming a shelter to the entrance door of a large building. The earliest known are the long arcaded porches in front of the early Christian basilicas, called Narthex. In later times they assume two forms—one, the projecting erection covering the entrance at the west front of cathedrals, and divided into three or more doorways, etc.; and the other, a kind of covered chambers open at the ends, and having small windows at the sides as a protection from rain.

Portal.—A name given to the deeply recessed and richly decorated entrance doors to the cathedrals in Continental Europe.

Portcullis.—A strong-framed grating of oak, the lower points shod with iron, and sometimes entirely made of metal, hung so as to slide up and down in grooves with counterbalances, and intended to protect the gateways of castles, etc.

Portico.—An open space before the door or other entrance to any building, fronted with columns. A portico is distinguished as prostyle or *in antis* according as it projects from or recedes within the building, and is further designated by the number of columns its front may consist of.

Post.—Square timbers set on end. The term is especially applied to those which support the corners of a building, and are framed into bressummers or crossbeams under the walls.

Posticum.—A portico behind a temple.

Presbytery.—A word applied to various parts of large churches in a very am-

biguous way. Some consider it to be the choir itself; others, what is now named the sacrarium. Traditionally, however, it seems to be applied to the vacant space between the back of the high altar and the entrance to the lady-chapel, as at Lincoln and Chichester; in other words, the back- or retro-choir.

Priming.—The laying on of the first shade of color, in oil paint, and generally consisting mostly of oil, to protect and fill the wood.

Priory.—A monastic establishment, generally in connection with an abbey, and presided over by a prior, who was a subordinate to the abbot, and held much the same relation to that dignitary as a dean does to a bishop.

Profile.—The outline; the contour of a part, or the parts composing an order, as of a base, cornice, etc.; also, the perpendicular section. It is in the just proportion of their profiles that the chief beauties of the different orders of architecture depend. The ancients were most careful of the profiles of their mouldings.

Proscenium.—The front part of the stage of ancient theatres, on which the actors performed.

Prostyle.—A portico in which the columns project from the building to which it is attached.

Protractor.—A mathematical instrument for laying down and measuring angles on paper, used in drawing or plotting.

Pseudo-dipteral.—False double-winged. When the inner row of columns of a dipteral arrangement is omitted and the space from the wall of the building to the columns is preserved, it is pseudo-dipteral.

Puddle.—To settle loose dirt by turning on water, so as to render it firm and solid.

Pugging.—A coarse kind of mortar laid on the boarding, between floor joists, to prevent the passage of sound; also called deafening.

Pulpit.—A raised platform with enclosed front, whence sermons, homilies, etc., were delivered. Pulpits were probably derived in their modern form from the ambones in the early Christian church. There are many old pulpits of stone, though the majority are of wood. Those in the churches are generally hexagonal or octagonal; and some stand on stone bases, and others on slender wooden stems, like columns. The designs vary according to the periods in which they were erected, having panelling, tracery, cuspings, crockets, and other ornaments then in use. Some are extremely rich, and ornamented with color and gilding. A few also have fine canopies or sounding boards. Their usual place is in the nave, mostly on the north side, against the second pier from the chancel arch. Pulpits for addressing the people in the open air were common in the Mediæval period, and stood near a road or cross. Thus, there was one at Spitalfields, and one at St. Paul's, London. External pulpits still remain at Magdalen College, Oxford, and at Shrewsbury, England.

Purlins.—Those pieces of timbers which support the rafters to prevent them from sinking.

Putlog.—Horizontal pieces for supporting the floor of a scaffold, one end being inserted into putlog holes, left for that purpose in the masonry.

Putty in Plastering.—Lump lime slacked with water to the consistency of cream, and then left to harden by evaporation till it becomes like soft putty. It is then mixed with plaster of Paris, or sand, for the finishing coat.

Puzzolana.—A grayish earth used for building under water.

Pyramid.—A solid, having one of its sides, called a base, a plane figure, and the other sides triangles, these points joining in one point at the top, called the vertex. Pyramids are called triangular, square, etc., according to the form of their bases.

Pyx.—In Roman Catholic churches, the box in which the host, or consecrated wafer, is kept.

Quadrangle.—A square or quadrangular court surrounded by buildings, as was often done formerly in monasteries, colleges, etc.

Quarry.—A pane of glass cut in a diamond or lozenge form.

Quarry-face.—Ashlar as it comes from the quarry, squared off for the joints only, with split face. In distinction from Rock-face, in that the latter may be weather-worn, while Quarry-face should be fresh split. The terms are often used indiscriminately.

Quatrefoil.—Any small panel or perforation in the form of a four-leaved flower. Sometimes used alone, sometimes in circles and over the aisle windows, but more frequently in square panels. They are generally cusped, and the cusps are often feathered.

Queen Truss.—A truss framed with two vertical tie-posts, in distinction from the king-post, which has but one. The upright ties are called Queen-posts.

Quirk Mouldings.—The convex part of Grecian mouldings when they recede at the top, forming a reëntrant angle, with the surface which covers the mouldings.

Quoins.—Large squared stones at the angles of buildings, buttresses, etc., generally used to stop the rubble or rough stone work, and that the angles may be true and stronger. Saxon quoin stones are said to have been composed of one long and one short stone alternately. Early quoins are generally roughly axed; in later times they had a draught tooled by the chisel round the outside edges, and later still were worked fine from the saw.

Rafters.—The joist to which the roof boarding is nailed. *Principal rafters* are the upper timbers in a truss, having the same inclination as the common rafters.

Rail.—A piece of timber or metal extending from one post to another, as in fences, balustrades, staircases, etc. In framing and panelling, the horizontal pieces are called rails, and the perpendicular, *stiles.*

Raking.—Mouldings whose arrises are inclined to the horizon.

Ramp.—A concavity on the upper side of hand railings formed over risers, made by a sudden rise of the steps above. Any concave bend or slope in the cap or upper member of any piece of ascending or descending workmanship.

Rampant.—A term applied to an arch whose abutments spring from an inclined plane.

Random Work.—A term used by stone-masons for stones fitted together at random without any attempt at laying them in courses. *Random Coursed Work* is a like term applied to work coursed in horizontal beds, but the stones are of any height, and fitted to one another.

Range Work.—Ashlar laid in horizontal courses; same as coursed ashlar.

Rebate.—A groove on the edges of a board.

Recess.—A depth of some inches in the thickness of a wall, as a niche, etc.

Refectory.—The hall of a monastery, convent, etc., where the religious took their chief meals together. It much resembled the great halls of mansions, castles, etc., except that there frequently was a sort of ambo, approached by steps, from which to read the Legenda Sanctorum, etc., during meals.

Reglet.—A flat, narrow moulding, used to separate from each other the parts or members of compartments and panels, to form frets, knots, etc.

Renaissance (a new birth).—A name given to the revival of Roman architecture which sprang into existence in Italy as early as the beginning of the fifteenth

century, and reached its zenith in that country at the close of the century. There are several divisions of this style as developed in different localities; viz.,

The Florentine Renaissance, of which the Pitti Palace, by Brunelleschi, is one of the best examples.

The Venetian Renaissance, characterized by its elegance and richness.

The Roman Renaissance, which originated in Rome, under the architects known as Bronte, Vignola, and Michael Angelo. Of this style the Farnese Palace, St. Peter's, and the modern Capitol at Rome are the best examples.

The French Renaissance, introduced into France in the latter part of the fifteenth century, by Italian architects, where it flourished until the middle of the seventeenth century. The Renaissance style was introduced into Germany about the middle of the sixteenth century, and into England about the same time by John of Padua, architect to Henry VIII. This style in England is generally known under the name of Elizabethan.

Rendering.—In drawing, finishing a perspective drawing in ink or color, to bring out the spirit and effect of the design. 2. The first coat of plaster on brick or stone work.

Reredos, Dorsal, or **Dossel.**—The screen or other ornamental work at the back of an altar. In some large English cathedrals, as Winchester, Durham, St. Albans, etc., this is a mass of splendid tabernacle work, reaching nearly to the groining. In smaller churches there are sometimes ranges of arcades or panellings behind the altars; but, in general, the walls at the back and sides of them were of plain masonry, and adorned with hangings or paraments. In the large churches of Continental Europe the high altar usually stands under a sort of canopy or ciborium, and the sacrarium is hung round at the back and sides with curtains on movable rods.

Reticulated Work.—That in which the courses are arranged in a form like the meshes of a net. The stones or bricks are square and placed lozenge-wise.

Return.—The continuation of a moulding, projection, etc., in an opposite direction.

Return Head.—One that appears both on the face and edge of a work.

Reveal.—The two vertical sides of an aperture, between the front of a wall and the window or door frame.

Rib.—A moulding or projecting piece upon the interior of a vault, or used to form tracery and the like. The earliest groining had no ribs. In early Norman times plain flat arches crossed each other, forming ogive ribs. These by degrees became narrower, had greater projection, and were chamfered. In later Norman work the ribs were often formed of a large roll placed upon the flat band, and then of two rolls side by side with a smaller roll or a fillet between them, much like the lower member. Sometimes they are enriched with zigzags and other Norman decorations, and about this time bosses became of very general use. As styles progressed, the mouldings were more undercut, richer, and more elaborate, and had the dog-tooth or ball-flower or other characteristic ornament in the hollows. In all instances the mouldings are of similar contours to those of arches, etc., of the respective periods. Later, wooden roofs are often formed into cants or polygonal barrel vaults, and in these the ribs are generally a cluster of rounds, and form square or stellar panels, with carved bosses or shields at the intersections.

Ridge.—The top of a roof which rises to an acute angle.

Ridge-pole.—The highest horizontal timber in a roof, extending from top to top of the several pairs of rafters of the trusses, for supporting the heads of the jack rafters.

Rilievo, or Relief.—The projection of an architectural ornament.

Rise.—The distance through which anything rises, as the rise of a stair, or inclined plane.

Riser.—The vertical board under the tread in stairs.

Rococo Style.—A name given to that variety of the Renaissance which was in vogue during the seventeenth and the latter part of the sixteenth century.

Romanesque Style.—The term Romanesque embraces all those styles of architecture which prevailed between the destruction of the Roman Empire and the beginning of Gothic architecture. In it are included the Early Roman Christian architecture, Byzantine, Mahometan, and the later Romanesque architecture proper, which was developed in Italy, France, England, and Germany. This later Romanesque, which was quite different from the preceding, came into vogue during the tenth century, and reached its height during the twelfth century, and in the thirteenth century gave way to the Pointed or Gothic style. In England, Romanesque architecture is known under the name of the Saxon, Norman, and Lombard styles, according to the different political periods.

Rood.—A name applied to a crucifix, particularly to those which were placed in the rood-loft or chancel screens. These generally had not only the image of the crucified Saviour, but also those of St. John and the Virgin Mary standing one on each side. Sometimes other saints and angels are by them, and the top of the screen is set with candlesticks or other decoration.

Rood-loft, Rood-screen, Rood-beam, Jube Gallery, etc.—The arrangement to carry the crucifix or rood, and to screen off the chancel from the rest of the church during the breviary services, and as a place whence to read certain parts of those services. Sometimes the crucifix is carried simply on a strong transverse beam, with or without a low screen, with folding-doors below but forming no part of such support. In European churches the general construction of wooden screens is close panelling beneath, about 3 feet to 3 feet 6 inches high, on which stands screen work composed of slender turned balusters or regular wooden mullions, supporting tracery more or less rich, with cornices, cresting, etc., and often painted in brilliant colors and gilded. These not only enclose the chancels, but also chapels, chantries, and sometimes even tombs. In English mansions, and some private houses, the great halls were screened off by a low passage at the end opposite to the dais, over which was a gallery for the use of minstrels or spectators. These screens were sometimes close and sometimes glazed.

Rood-tower.—A name given by some writers to the central tower, or that over the intersection of the nave and chancel with the transepts.

Roof.—The covering or upper part of any building.

Roofing.—The material put on a roof to make it water-tight.

Rose Window.—A name given to a circular window with radiating tracery; called also wheel window.

Rostrum.—An elevated platform from which a speaker addresses an audience.

Rotunda.—A building which is round both within and without. 2. A circular room under a dome in large buildings is also called the rotunda.

Roughcast.—A sort of external plastering in which small sharp stones are mixed, and which, when wet, is forcibly thrown or cast from a trowel against the wall, to which it forms a coating of pleasing appearance. Roughcast work has been used in Europe for several centuries, where it was much used in timber houses, and when well executed the work is sound and durable. The mortar for roughcast work should always have cement mixed with it.

Rubble Work.—Masonry of rough, undressed stones. When only the roughest irregularities are knocked off, it is called scabbled rubble, and when the

stones in each course are rudely dressed to nearly a uniform height, ranged rubble.

Rudenture.—The figure of a rope or staff, which is frequently used to fill up the flutings of columns, the convexity of which contrasts with the concavity of the flutings, and serves to strengthen the edges. Sometimes, instead of a convex shape, the flutings are filled with a flat surface ; sometimes they are ornamentally carved, and sometimes on pilasters, etc. Rudentures are used in relief without flutings, as their use is to give greater solidity to the lower part of the shaft, and secure the edges. They are generally only used in columns which rise from the ground and are not to reach above one-third of the height of the shaft.

Rustic or **Rock Work.**—A mode of building in imitation of nature. This term is applied to those courses of stone work the face of which is jagged or picked so as to present a rough surface. That work is also called rustic in which the horizontal and vertical channels are cut in the joinings of stones, so that when placed together an angular channel is formed at each joint. *Frosted rustic work* has the margins of the stones reduced to a plane parallel to the plane of the wall, the intermediate parts having an irregular surface. *Vermiculated rustic work* has these intermediate parts so worked as to have the appearance of having been eaten by worms. *Rustic chamfered work*, in which the face of the stones is smooth, and parallel to the face of the wall, and the angles bevelled to an angle of one hundred and thirty-five degrees with the face so that two stones coming together on the wall, the bevelling will form an internal right angle.

Sacristy.—A small chamber attached to churches, where the chalices, vestments, books, etc., were kept by the officer called the sacristan. In the early Christian basilicas there were two semicircular recesses or apsides, one on each side of the altar. One of these served as a sacristy, and the other as the bibliotheca or library. Some have supposed the sacristy to have been the place where the vestments were kept, and the vestry that where the priests put them on ; but we find from Durandus that the sacrarium was used for both these purposes. Sometimes the place where the altar stands enclosed by the rails has been called sacrarium.

Saddle Bars.—Narrow horizontal iron bars passing from mullion to mullion, and often through the whole window, from side to side, to steady the stone work, and to form stays, to which the lead work is secured. When the bays of the windows are wide, the lead lights are further strengthened by upright bars passing through eyes forged on the saddle bars, and called stanchions. When saddle bars pass right through the mullions in one piece, and are secured to the jambs, they have sometimes been called stay bars.

Sagging.—The bending of a body in the middle by its own weight, or the load upon it.

Salient.—A projection.

Salon.—A spacious and elegant apartment for the reception of company, or for state purposes, or for the reception of paintings, and usually extending through two stories of the house. It may be square, oblong, polygonal, or circular.

Sanctuary.—That part of a church where the altar is placed ; also, the most sacred or retired part of a temple. 2. A place for divine worship ; a church.

Sanctus Bell-cot, or **Turret.**—A turret or enclosure to hold the small bell sounded at various parts of the service, particularly where the words "Sanctus," etc., are read. This differs but little from the common bell-cot, except that it is generally on the top of the arch dividing the nave from the chancel. Sometimes,

however, the bell seems to have been placed in a cot outside the wall. In England sanctus bells have also been placed over the gables of porches. In Continental Europe they run up into a sort of small slender spire, called *flèche* in France, and *guglio* in Italy.

Saracenic Architecture.—That Eastern style employed by the Saracens, and which distributed itself over the world with the religion of Mahomet. It is a modification and combination of the various styles of the countries which they conquered.

Sarcophagus.—A tomb or coffin made of stone, and intended to contain the body.

Sash.—The framework which holds the glass in a window.

Scagliola.—An imitation of colored marbles in plaster work, made by a combination of gypsum, glue, isinglass, and coloring matter, and finished with a high polish, invented between 1600 and 1649.

Scabble.—To dress off the rougher projections of stones for rubble masonry with a stone axe or scabbling hammer.

Scantling.—The dimensions of a piece of timber in breadth and thickness; also, studding for a partition, when under five inches square.

Scarfing.—The joining and bolting of two pieces of timber together transversely, so that the two appear as one.

Sconce.—A fixed hanging or projecting candlestick.

Scotia.—A concave moulding, most commonly used in bases, which projects a deep shadow on itself, and is thereby a most effective moulding under the eye, as in a base. It is like a reversed ovolo, or, rather, what the mould of an ovolo would present.

Scratch Coat.—The first coat of plaster, which is scratched to afford a bond for the second coat.

Screeds.—Long narrow strips of plaster put on horizontally along a wall, and carefully faced out of wind, to serve as guides for plastering the wide intervals between them.

Screen.—Any construction subdividing one part of a building from another, as a choir, chantry, chapel, etc. The earliest screens are the low marble podia shutting off the chorus cantantium in the Roman basilicas, and the perforated cancelli enclosing the bema, altar, and seats of the bishops and presbyters. The chief screens in a church are those which enclose the choir or the place where the breviary services are recited. In Continental Europe this is done not only by doors and screen work, but also, when these are of open work, by curtains, the laity having no part in these services. In England screens were of two kinds: one, of open wood-work, generally called rood-screens or jubes, and which the French call *grilles, clôtures du chœur;* the other, massive enclosures of stone work enriched with niches, tabernacles, canopies, pinnacles, statues, crestings, etc., as at Canterbury, York, Gloucester, and many other places.

Scribing.—Fitting wood-work to an irregular surface.

Section.—A drawing showing the internal heights of the various parts of a building. It supposes the building to be cut through entirely, so as to exhibit the walls, the heights of the internal doors and other apertures, the heights of the stories, thicknesses of the floors, etc. It is one of the species of drawings necessary to the exhibition of a Design.

Sedilia.—Seats used by the celebrants during the pauses in the mass. They are generally three in number—for the priest, deacon, and sub-deacon—and are in England almost always a species of niches cut into the south walls of churches, separated by shafts or by a species of mullions, and crowned with canopies, pinnacles, and other enrichments more or less elaborate. The piscina and

ambry sometimes are attached to them. In Continental Europe the sedilia are often movable seats; a single stone seat has rarely been found.

Set-off.—The horizontal line shown where a wall is reduced in thickness, and, consequently, the part of the thicker portion appears projecting before the thinner. In plinths this is generally simply chamfered. In other parts of work the set-off is generally concealed by a projecting string. Where, as in parapets, the upper part projects before the lower, the break is generally hid by a corbel table. The portions of buttress caps which recede one behind another are also called set-offs.

Shaft.—In Classical architecture that part of a column between the necking and the apophyge at the top of the base. In later times the term is applied to slender columns either standing alone or in connection with pillars, buttresses, jambs, vaulting, etc.

Shed Roof, or **Lean-to.**—A roof with only one set of rafters, falling from a higher to a lower wall, like an aisle roof.

Shore.—A piece of timber placed in an oblique direction to support a building or wall temporarily while it is being repaired or altered.

Shrine.—A sort of ark or chest to hold relics. It is sometimes merely a small box, generally with a raised top like a roof; sometimes an actual model of churches; sometimes a large construction, like that of Edward the Confessor at Westminster, of St. Genevieve at Paris, etc. Many are covered with jewels in the richest way; that of San Carlo Borromeo, at Milan, is of beaten silver.

Sills.—Are the timbers on the ground which support the posts and superstructure of a timber building. The term is most frequently applied to those pieces of timber or stone at the bottom of doors or windows.

Skewback.—The inclined stone from which an arch springs.

Skirtings.—The narrow boards which form a plinth around the margin of a floor, now generally called the base.

Sleeper.—A piece of timber laid on the ground to receive floor joists.

Soffit.—The lower horizontal face of anything, as, for example, of an entablature resting on and lying open between the columns or the under face of an arch where its thickness is seen.

Sound Board.—The covering of a pulpit to deflect the sound into a church.

Spall.—Bad or broken brick: stone chips.

Span.—The distance between the supports of a beam, girder, arch, truss, etc.

Spandrel, or **Spandril.**—The space between any arch or curved brace and the level label, beams, etc., over the same. The spandrels over doorways in Perpendicular works are generally richly decorated.

Specification.—Architect's. The designation of the kind, quality, and quantity of work and material to go in a building, in conjunction with the working drawings.

Spire.—A sharply pointed pyramid or large pinnacle, generally octagonal in England, and forming a finish to the tops of towers. Timber spires are very common in England. Some are covered with lead in flat sheets, others with the same metal in narrow strips laid diagonally. Very many are covered with shingles. In Continental Europe there are some elegant examples of spires of open timber work covered with lead.

Splayed.—The jamb of a door, or anything else of which one side makes an oblique angle with the other.

Springer.—The stone from which an arch springs: in some cases this is a capital, or impost; in other cases the mouldings continue down the pier. The lowest stone of the gable is sometimes called a springer.

Squinches.—Small arches or corbelled set-offs running diagonally and, as it

were, cutting off the corners of the interior of towers, to bring them from the square to the octagon, etc., to carry the spire.

Squint.—An oblique opening in the wall of a church; especially, in mediæval architecture, an opening so placed as to afford a view of the high altar from the transept or aisles.

Staging.—A structure of posts and boards for supporting workmen and material in building.

Stall.—A fixed seat in the choir for the use of the clergy. In early Christian times the thronus cathedra, or seat of the bishop, was in the centre of the apsis or bema behind the altar, and against the wall; those of the presbyters also were against the wall, branching off from side to side around the semicircle. In later times the stalls occupied both sides of the choir, return seats being placed at the ends for the prior, dean, precentor, chancellor, or other officers. In general, in cathedrals, each stall is surmounted by tabernacle work, and rich canopies, generally of oak.

Stanchion.—A word derived from the French *étançon*, a wooden post, applied to the upright iron bars which pass through the eyes of the saddle bars or horizontal irons to steady the lead lights. The French call the latter *traverses*, the stanchions *montants*, and the whole arrangement *armature*. Stanchions frequently finish with ornamental heads forged out of the iron.

Steeple.—A general name for the whole arrangement of tower, belfry, spire, etc.

Stereobate.—A basement, distinguished from the nearly equivalent term stylobate by the absence of columns.

Stile.—The upright piece in framing or panelling.

Stilted.—Anything raised above its usual level. An arch is stilted when its centre is raised above the line from which the arch appears to spring.

Stoop.—A seat before the door; often a porch with a balustrade and seats on the sides.

Stoup.—A basin for holy water at the entrance of Roman Catholic churches, into which all who enter dip their fingers and cross themselves.

Straight Arch.—A form of arch in which the intrados is straight, but with its joints radiating as in a common arch.

Strap.—An iron plate for connecting two or more timbers, to which it is screwed by bolts. It generally passes around one of the timbers.

Stretcher.—A brick or block of masonry laid lengthwise of a wall.

String Board.—A board placed next to the well-hole in wooden stairs, terminating the ends of the steps. The string piece is the piece of board put under the treads and risers for a support, and forming the support of the stair.

String-course.—A narrow, vertically faced and slightly projecting course in an elevation. If window-sills are made continuous, they form a string-course: but if this course is made thicker or deeper than ordinary window-sills, or covers a set-off in the wall, it becomes a blocking-course. Also, horizontal mouldings running under windows, separating the walls from the plain part of the parapets, dividing towers into stories or stages, etc. Their section is much the same as the labels of the respective periods; in fact, these last, after passing round the windows, frequently run on horizontally and form strings. Like labels, they are often decorated with foliages, ball-flowers, etc.

Studs, or Studding.—The small timbers used in partitions and outside wooden walls, to which the laths and boards are nailed.

Style.—The term style in architecture has obtained a conventional meaning beyond its simpler one, which applies only to columns and columnar arrangements. It is now used to signify the differences in the mouldings, general out-

lines, ornaments, and other details which exist between the works of various nations, and also those differences which are found to exist between the works of any nation at different times.

Stylobate.—A basement to columns. Stylobate is synonymous with pedestal, but is applied to a continued and unbroken substructure or basement to columns, while the latter term is confined to insulated supports. The Greek temples generally had three or more steps all around the temple, the base of the column resting on the top step ; this was the stylobate.

Subsellium.—A name sometimes given to the seat in the stalls of churches ; same as miserere.

Summer.—A girder or main-beam of a floor ; if supported on two-story posts and open below, it is called a Brace-summer.

Surbase.—A cornice or series of mouldings on the top of the base of a pedestal, podium, etc.; a moulding above the base.

Surface.—To make plane and smooth.

Systyle.—An intercolumniation to which two diameters are assigned.

Tabernacle.—A species of niche or recess in which an image may be placed. They are generally highly ornamented and often surmounted with crocketed gables. The word tabernacle is also often used to denote the receptacle for relics, which was often made in the form of a small house or church.

Tabernacle Work.—The rich ornamental tracery forming the canopy, etc., to a tabernacle, is called tabernacle work ; it is common in the stalls and screens of cathedrals, and in them is generally open or pierced through.

Tail Trimmer.—A trimmer next to the wall, into which the ends of joists are fastened to avoid flues.

Tamp.—To pound the earth down around a wall after it has been thrown in.

Tapestry.—A kind of woven hangings of wool or silk, ornamented with figures, and used formerly to cover and adorn the walls of rooms. They were often of the most costly materials and beautifully embroidered.

Temple.—An edifice destined, in the earliest times, for the public exercise of religious worship.

Templet, or **Template.**—A mould used by masons for cutting or setting work. 2. A short piece of timber sometimes laid under a girder.

Terminal.—Figures of which the upper parts only, or perhaps the head and shoulders alone, are carved, the rest running into a parallelopiped, and sometimes into a diminishing pedestal, with feet indicated below, or even without them, are called terminal figures.

ANCIENT TERMINI.

Terra-cotta.—Baked clay of a fine quality. Much used for bas-reliefs for adorning the friezes of temples. In modern times employed for architectural ornaments, statues, vases, etc.

Tessellated Pavements.—Those formed of tesseræ, or, as some write it, tessellæ, or small cubes from half an inch to an inch square, like dice, of pottery, stone, marble, enamel, etc.

Tetrastyle.—A portico of four columns in front.

Tholobate.—That on which a dome or cupola rests. This is a term not in general use, but it is not the less of useful application. What is generally termed the attic above the peristyle and under the cupola of St. Paul's, London, would be correctly desig-

nated the tholobate. A tholobate of a different description, and one to which no other name can well be applied, is the circular substructure to the cupola of the University College, London.

Throat.—A channel or groove made on the under-side of a string-course, coping, etc., to prevent water from running inward toward the walls.

Tie.—A timber, rod, chain, etc., binding two bodies together, which have a tendency to separate or diverge from each other. The *tie-beam* connects the bottom of a pair of principal rafters, and prevents them from bursting out the wall.

Tiles.—Flat pieces of clay burned in kilns, to cover roofs in place of slates or lead. 2. Also, flat pieces of burned clay, either plain or ornamented, glazed or unglazed, used for floors, wainscoting, and about fireplaces, etc. 3. Small square pieces of marble are also called tile.

Tongue.—The part of a board left projecting, to be inserted into a groove.

Tooth Ornament.—One of the peculiar marks of the Early English period of Gothic architecture, generally inserted in the hollow mouldings of doorways, windows, etc.

Torso.—A mutilated statue of which nothing remains but the trunk. Columns with twisted shafts have also this term. Of this kind there are several varieties.

Torus.—A protuberance or swelling, a moulding whose form is convex, and generally nearly approaches a semicircle. It is most frequently used in bases, and is generally the lowest moulding in a base.

TORUS.

Tower.—An elevated building originally designed for purposes of defence. Those buildings are of the remotest antiquity, and are, indeed, mentioned in the earliest Scriptures. In mediæval times they were generally attached to churches, to cemeteries, to castles, or used as bell-towers in public places of large cities. In churches, the towers of the Saxon period were generally square. Norman towers were also generally square. Many were entirely without buttresses; others had broad, flat, shallow projections which served for this purpose. The lower windows were very narrow, with extremely wide splays inside, probably intended to be defended by archers. The upper windows, like those of the preceding style, were generally separated into two lights, but by a shaft or short column, and not by a baluster. Early English towers were generally taller, and of more elegant proportions. They almost always had large projecting buttresses, and frequently stone staircases. The lower windows, as in the former style, were frequently mere arrow-slits; the upper were in couplets or triplets, and sometimes the tower top had an arcade all round. The spires were generally broach spires; but sometimes the tower tops finished with corbel courses and plain parapets, and (rarely) with pinnacles. There are a few Early English towers which break into the octagon from the square toward the top, and still fewer which finish with two gables. Both these methods of termination, however, are common in Continental Europe. At Vendôme, Chartres, and Senlis the towers have octagonal upper stages surrounded with pinnacles, from which elegant spires arise. In the North of Italy, and in Rome, they are generally tall square shafts in four to six stages, without buttresses, with couplets or triplets of semicircular windows in each stage, generally crenellated at top, and covered with a low pyramidal roof. The well-known leaning tower at Pisa is cylindrical, in five stories of arcaded colonnades. In Ireland there are in some of the churchyards very curious round towers.

Tracery.—The ornamental filling in of the heads of windows, panels, circular windows, etc., which has given such characteristic beauty to the architecture of

the fourteenth century. Like almost everything connected with mediæval architecture, this elegant and sometimes fairy-like decoration seems to have sprung from the smallest beginnings. The circular-headed window of the Normans gradually gave way to the narrow-pointed lancets of the Early English period, and, as less light was afforded by the latter system than by the former, it was necessary to have a greater number of windows; and it was found convenient to group them together in couplets, triplets, etc. When these couplets were assembled under one label, a sort of vacant space or spandrel was formed over the lancets and under the label. To relieve this, the first attempts were simply to perforate this flat spandrel, first by a simple lozenge-shaped or circular opening, and afterward by a quatrefoil. By piercing the whole of the vacant spaces in the window head, carrying mouldings around the tracery, and adding cusps to it, the formation of tracery was complete, and its earliest result was the beautiful geometrical work such as is found at Westminster Abbey.

Transept.—That portion of a church which passes transversely between the nave and choir at right angles, and so forms a cross on the plan.

Transom.—The horizontal construction which divides a window into heights or stages. Transoms are sometimes simple pieces of mullions placed transversely as cross-bars, and in later times are richly decorated with cuspings, etc.

Traverse.—To plane in a direction across the grain of the wood, as to traverse a floor by planing across the boards.

Tread.—The horizontal part of a step of a stair.

Trefoil.—A cusping the outline of which is derived from a three-leaved flower or leaf, as the quatrefoil and cinque-foil are from those with four and five.

Trellis.—Lattice-work of metal or wood for vines to run on.

Trestle.—A movable frame or support for anything; when made of a cross piece with four legs it is called by carpenters a horse.

Triforium.—The arcaded story between the lower range of piers and arches and the clere-story. The name has been supposed to be derived from *tres* and *fores*—three doors, or openings—that being a frequent number of arches in each bay.

Triglyph.—The vertically channelled tablets of the Doric frieze are called triglyphs, because of the three angular channels in them—two perfect and one divided—the two chamfered angles or hemiglyphs being reckoned as one. The square sunk spaces between the triglyphs on a frieze are called metopes.

Trim.—Of a door, sometimes used to denote the locks, knobs, and hinges.

Trimmer.—The beam or floor joist into which a header is framed.

Trimmer Arch.—An arch built in front of a fireplace, in the thickness of the floor, between two trimmers. The bottom of the arch starting from the chimney and the top pressing against the header.

Tuck-pointing.—Marking the joints of brickwork with a narrow parallel ridge of fine putty.

Tudor Style.—The architecture which prevailed in England during the reign of the Tudors; its period is generally restricted to the end of the reign of Henry VIII.

Turret.—A small tower, especially at the angles of larger buildings, sometimes overhanging and built on corbels, and sometimes rising from the ground.

Tuscan Order.—The plainest of the five orders of Classic architecture.

Tympanum.—The triangular recessed space enclosed by the cornice which bounds a pediment. The Greeks often placed sculptures representing subjects connected with the purposes of the edifice in the tympana of temples, as at the Parthenon and Ægina.

Under-croft.—A vaulted chamber under ground.

Upset.—To thicken, and shorten as by hammering a heated bar of iron on the end.

Vagina.—The upper part of the shaft of a terminus, from which the bust or figure seems to rise.

Valley.—The internal angle formed by two inclined sides of a roof.

Valley Rafters.—Those which are disposed in the internal angle of a roof to form the valleys.

Vane.—The weathercock on a steeple. In early times it seems to have been of various forms, as dragons, etc.; but in the Tudor period the favorite design was a beast or bird sitting on a slender pedestal, and carrying an upright rod, on which a thin plate of metal is hung like a flag, ornamented in various ways.

Vault.—An arched ceiling or roof. A vault is, indeed, a laterally conjoined series of arches. The arch of a bridge is, strictly speaking, a vault. Intersecting vaults are said to be groined. See *Groined Vaulting* for fuller description of vaults.

Verge.—The edge of the tiling, slate or shingles, projecting over the gable of a roof, that on the horizontal portion being called eaves.

Verge Board.—Often corrupted into Barge Board; the board under the verge of gables, sometimes moulded, and often very richly carved, perforated, and cusped, and frequently having pendants, and sometimes finials, at the apex.

Vermiculated.—Stones, etc., worked so as to have the appearance of having been worked by worms.

VERMICULATED.

Vestibule.—An anti-hall, lobby, or porch.

Vestry.—A room adjoining a church, where the vestments of the minister are kept and parish meetings held. In American Protestant churches, the Sunday-school room is often called the vestry.

Viaduct.—A structure of considerable magnitude, and usually of masonry, for carrying a railway across a valley.

Vignette.—A running ornament, representing, as its name imports, a little vine, with branches, leaves, and grapes. It is common in the Tudor period, and runs or roves in a large hollow or casement. It is also called Trayle.

Villa.—A country house for the retreat of the rich.

Volute.—The convolved or spiral ornament which forms the characteristic of the Ionic capital. Volute, scroll, helix, and cauliculus are used indifferently for the angular horns of the Corinthian capital.

Voussoir.—One of the wedge-like stones which form an arch; the middle one is called the key-stone.

Wainscot.—The wooden lining of walls, generally in panels.

Wall Plates.—Pieces of timber which are placed on top of brick or stone walls so as to form the support to the roof of a building.

Warped.—Twisted out of shape by seasoning.

Water Table.—A slight projection of the lower masonry or brickwork on the outside of a wall a few feet above the ground as a protection against rain.

Weather Boarding.—Boards lapped over each other to prevent rain, etc., from passing through.

Weathering.—A slight fall on the top of cornices, window-sills, etc., to throw off the rain.

Wicket.—A small door opening in a larger. They are common in mediæval doors, and were intended to admit single persons, and guard against sudden surprises.

Wind.—A turn, a bend. A wall is *out of wind* when it is a perfectly flat surface.

Wing.—A side building less than the main building.

Withes.—The partition between two chimney flues in the same stack.

ARCHITECTURAL TERMS AS DEFINED IN VARIOUS BUILDING LAWS,

COMPILED BY THE AMERICAN ARCHITECT AND BUILDING NEWS, PAGE 150, VOL. XXXIII.

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TERMS DEFINED.

[*The following terms chance to be defined in sundry building codes—which are mentioned in each case. The fact that other codes are not mentioned is not necessarily a proof that the term is not also elsewhere in use as defined.*]

Adjoining Owner.—The owner of the premises adjoining those on which work is doing or to be done. [*District of Columbia.*]

Alteration.—Any change or addition except necessary repairs in, to, or upon any building affecting an external, party, or partition wall, chimney, floor, or stairway, and "to alter" means to make such change or addition. [*Boston and Denver.*]

Appendages.—Dormer-windows, cornices, mouldings, bay-windows, towers, spires, ventilators, etc. [*Chicago, Minneapolis.*]

Areas.—Sub-surface excavations adjacent to the building-line for lighting or ventilation of cellars or basements. [*District of Columbia.*]

Attic Story.—A story situated either in whole or in part in the roof. [*Denver and District of Columbia.*]

Base.—"The *base* of a brick wall" means the course immediately above the foundation wall. [*Cincinnati and Cleveland.*]

Basement Story.—One whose floor is 12″ or more below the sidewalk, and whose height does not exceed 12′ in the clear; all such stories that exceed 12′ high shall be considered as first stories. [*Chicago, Louisville.*]

A story whose floor is 12″ or more below the grade of sidewalk. [*Milwaukee.*]

A story whose floor is 3′ or more below the sidewalk, and whose height does not exceed 11′ in the clear; all such stories that exceed 11′ high shall be considered as first stories. [*Minneapolis.*]

A story suitable for habitation, partially below the level of the adjoining street or ground.[1] [*District of Columbia and Denver.*]

(See **Cellar.**)

Bay-window.—A first-floor projection for a window other than a tower-projection or show-window. [*District of Columbia.*]

Any projection for a window other than a show-window. [*Denver.*]

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[1] And below the first floor of joists. [*District of Columbia.*]

Bearing Walls.—Those on which beams, trusses, or girders rest. [*New York and San Francisco.*]

Brick Building.—A building the walls of which are built of brick, stone, iron, or other substantial and incombustible materials. [*Boston, Denver, and Kansas City.*]

Building.—Any construction within the scope and purview of these regulations. [*District of Columbia.*]

Building Line.—The line of demarcation between public and private space. [*District of Columbia.*]

Building Owner.—The owner of premises on which work is doing or to be done. [*District of Columbia.*]

Business buildings shall embrace all buildings used principally for business purposes, thus including, among others, hotels, theatres, and office-buildings. [*Chicago, Louisville, Milwaukee, and Minneapolis.*]

Cellar.—Basement or lower story of any building, of which one-half or more of the height from the floor to the ceiling is below the level of the street[1] adjoining.[2] [*Boston, Denver, and Kansas City.*]

Portion of building below first floor of joists, if partially or entirely below the level of the adjoining parking, street, or ground, and not suitable for habitation. [*District of Columbia.*]

Cement-mortar.—A proper proportion of cement and sand without the admixture of lime. [*Kansas City.*]

Division Wall.—One that separates part of any building from another part of the same building. [*Cincinnati and Cleveland.*]

Floor-bearing walls extending through buildings from front to rear, and separating stores and tenements in buildings or blocks owned by the same party. [*Minneapolis.*]

(See **Partition-wall.**)

Dwelling-house Class.—All buildings except public buildings and buildings of the warehouse class. [*Cincinnati and Cleveland.*]

Shall not apply to buildings accommodating more than three families. [*San Francisco.*]

External Wall.—Every outer wall or vertical enclosure of a building other than a party-wall. [*Boston, Cincinnati, Cleveland, Denver, District of Columbia, Kansas City, and Providence.*]

First Story.—The story the floor of which is at or first above the level of the sidewalk or adjoining ground, the other stories to be numbered in regular succession, counting upward. [*Denver and District of Columbia.*]

Footing Course.—A projecting course or courses under base of foundation wall. [*Cincinnati and Cleveland.*]

Foundation.—That portion of wall below level of street curb,[3] and, where the wall is not on a street, that portion of wall below the level of the highest ground next to the wall. [*Boston, Kansas City, New York, and Providence.*]

Portion of exterior wall below surface of adjoining earth or pavement, and portion of partition or party wall below level of basement or cellar floor. [*District of Columbia and Denver.*]

Foundation, Basement, or Cellar Walls.—That part of walls of building that are below the floor or joists, which are on or next above the grade line. [*Detroit.*]

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[1] Ground. [*Providence.*]

[2] And not suitable for habitation. [*Denver.*]

[3] "And serve as supports for piers, columns, girders, beams, or other walls." [*New York.*]

Portion of the wall below the level of street curb, in front of the central line of building. [*San Francisco.*]

Incombustible scantling partition.—One plastered on both sides upon iron lath or wire cloth, and filled in with brickwork 8″ high from floor, provided the building is not over 80′ high. [*Chicago.*]

Incombustible Roofing.—Covered with not less than three (3) thicknesses roofing-felt, and good coat of tar and gravel, or with tin, corrugated-iron, or other fire-resisting material with standing-seam or lap-joint. [*Denver.*]

Lengths.—Walls are deemed to be divided into distinct *lengths* by return walls, and the length of every wall is measured from the centre of one return wall to the centre of another, provided that such return walls are external or party cross-walls of the thickness herein required, and bonded into the walls so deemed to be divided. [*Cincinnati and Cleveland.*]

Inflammable Material.—Dry goods, clothing, millinery, and the like in stores, flyings or goods in factories, or other substance readily ignited by droppings or flyings from electric lights. [*Minneapolis.*]

Lodging-house.—A building in which persons are temporarily accommodated with sleeping [1] apartments, and includes hotels. [*Boston and Kansas City.*]

Any building or portion thereof in which persons are lodged for hire for less than a week at one time. [*District of Columbia and Providence.*]

Any building or portion thereof in which persons are lodged for hire temporarily, and includes hotels. [*Denver.*]

Mansard Roof.—One formed with an upper and under set of rafters, the upper set more inclined to the horizon than the lower set. [*Denver and District of Columbia.*]

Oriel Window.—A projection for a window above the first floor. [*District of Columbia.*]

Partition.—An interior division constructed of iron, glass, wood, lath and plaster, or other destructible natures. [*District of Columbia.*]

Partition-wall.—Any interior wall of masonry in a building. [*Boston, Kansas City, and Providence.*]

An interior wall of non-combustible material. [*District of Columbia.*]

Any interior division constructed of iron, glass, wood, lath and plaster, or any combination of those materials. [*Denver.*]

(See **Division Wall.**)

Party-wall.—Every wall used, or built, in order to be used, as a separation of two or more buildings.[2] [*Boston, Cincinnati, Cleveland, Denver, Kansas City, and Providence.*]

A wall built upon dividing line between adjoining premises for their common use. [*District of Columbia.*]

Parking.—The space between the sidewalk and the building line. [*District of Columbia.*]

Parking Line.—The line separating parking and sidewalk. [*District of Columbia.*]

Public Building.—Every building used as church, chapel, or other place of public worship; also every building used as a college, school, public hall, hospital, theatre, public concert-room, public ball-room, public lecture-room, or for any public assemblage. [*Boston, Chicago, Cincinnati, Cleveland, Denver, Kansas City, and Minneapolis.*]

Such buildings as shall be owned and occupied for public purposes for this

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[1] Staying apartments. [*Kansas City.*]

[2] To be used jointly by separate buildings. [*Cincinnati and Cleveland.*]

State, the United States, the corporation of the City of Brooklyn, or other public schools within said city. [*Brooklyn.*]

Public Hall.—Every theatre, opera-house, hall, church, school, or other building intended to be used for public assemblage. [*Milwaukee and Louisville.*]

Return Wall.—No wall subdividing any building shall be deemed a **return wall**, as before mentioned, unless it is two-thirds the height of the external or party-walls. [*Cincinnati and Cleveland.*]

Shed.—A skeleton structure for storage or shelter. [*District of Columbia.*]

Open structure, enclosed only on one side and end, and erected on the ground. [*San Francisco.*]

Open or closed board structure. [*Denver.*]

Show-window.—A store-window in which goods are displayed for sale or advertisement. [*District of Columbia and Denver.*]

Square thereof.—The square or level of the walls before commencing the pitch for roof. [*District of Columbia.*]

Standard Depth for Foundations.—For brick and stone buildings, 14′ below curb line. [*San Francisco.*]

Standard Depth of Cellars.—16′, measured down from sidewalk grade at property line. [*Memphis.*]

Standard Iron Door.—Made of No. 12 plate-iron, frame or continuous 12″ x 2″ x ⅜″ angle-iron, firmly riveted. Two panel doors, to have proper cross-bars, one panel on either side, fastened together with hooks or proper bolts top and bottom, and with not less than two lever-bars. All doors hung on iron frames of ⅜″ x 4″ iron, securely bolted together through wall, swung on three hinges, fitting close to frame all around; sill between doors, iron, brick, or stone, to rise not less than two (2) inches above floor on each side of opening. Lintel over door, brick, iron, or stone. Floors of basement, when doors are to swing, stone or cement, in no case wood. [*Denver.*]

Standard Skylight.—Constructed of wrought-iron frames, with hammered or desk-light glass not less than ½″ thick; not larger than 10′ by 12′, except by special permission of the Inspector. [*Denver.*]

Storehouse.—(See **Warehouse Class.**)

Street.—All streets, avenues, and public alleys. [*Minneapolis.*]

Tenement-house.—A building which, or any portion of which, is to be occupied, or is occupied, as a dwelling by more than three [1] families living independently of one another, and doing their cooking upon the premises. [*Boston, Denver, and Kansas City.*]

Or by more than two families [2] above the second floor, so living and cooking. [*Boston and Kansas City.*]

Building which shall contain more than two rooms in front on each floor, or which shall be built with a passage or arched way between distinct parts of the same building, or which building shall be intended for the separate accommodation of different families or occupants. [*Charleston.*]

Theatre.—Public hall containing movable scenery or fixed scenery which is not made of metal, plaster, or other incombustible material. [*Chicago, Louisville, and Milwaukee.*]

Thickness of a Wall.—The minimum thickness of such wall. [3] [*Boston, Cincinnati, Cleveland, Kansas City, Milwaukee, and Providence.*]

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[1] *Two* instead of three. [*District of Columbia and Minneapolis.*]

[2] Upon one floor, but having a common right in the halls, stairways, yards, etc. [*Providence.*]

[3] As applied to solid walls. [*Minneapolis and Providence.*]

Tinned Covered Fire-door.—Wood doors or shutters, double thickness of wood, cross or diagonal construction, covered on both sides and all edges with sheet-tin, joints securely clinched and nailed. [*Denver.*]

Tower Projection.—A projection designed for an ornamental door-entrance, for ornamental windows, or for buttresses. [*District of Columbia.*]

Vault.—An underground construction beneath parking or sidewalk. [*District of Columbia.*]

Veneered Building.—Frame structure, the walls covered above the sill by a 4′ wall of brick, instead of clapboards. [*Common understanding in Chicago, Milwaukee, and Minneapolis, but not defined by law.*]

Warehouse Class.—Buildings used for the storage of merchandise, manufactories in which machinery is operated, breweries, and distilleries. [*Cincinnati and St. Louis.*]

Width of buildings shall be computed by the way the beams are placed; the lengthwise of the beams shall be considered and taken to be the widthwise of the building. [*New York and San Francisco.*]

Wholesale store, or storehouse, shall embrace all buildings used (or intended to be used) exclusively for purpose of mercantile business or storage of goods. [*Chicago, Louisville, and Milwaukee.*]

Wooden Building.—A wooden or frame[1] building. [*Boston, Kansas City, and Minneapolis.*]

Any building of which an external or party wall is constructed in whole or in part of wood. [*Denver and District of Columbia.*]

Having more wood on the outside than that required for the door and window frames, doors, shutters, sash porticos, and wooden steps, and all frame buildings or sheds, although the sides and ends are proposed to be covered with corrugated iron or other metal, shall be deemed a wooden building under this law. [*Charleston and Nashville.*]

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[1] Or veneered. [*Minneapolis.*]

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CLASSIFIED LIST OF ADVERTISEMENTS.

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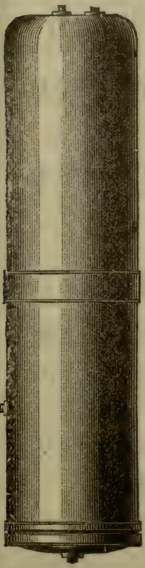

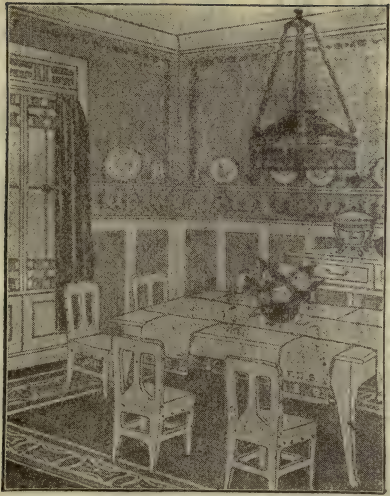

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